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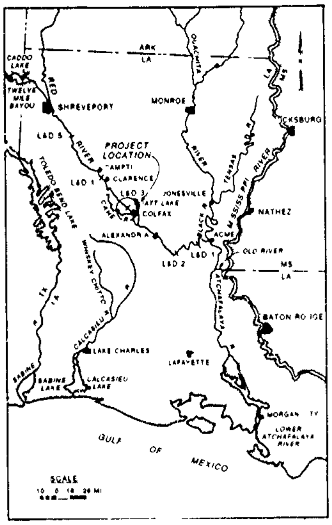
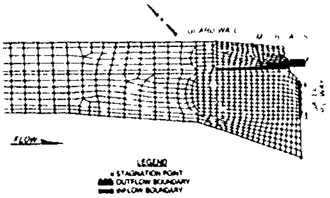
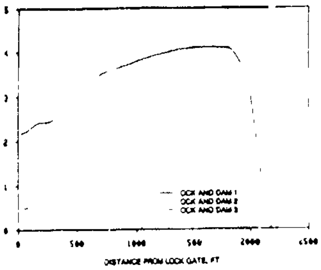
RED RIVER WATERWAY, LOCK AND DAM NO. 3

Report 5
SEDIMENTATION IN LOCK APPROACHES

TABS-2 Numerical Model Investigation







by

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13. ABSTRACT *(Maximum 200 words)*

A two-dimensional numerical model, TABS-2, was used to predict fine sediment deposition in the lock approach channels upstream and downstream from Lock and Dam No. 3 on the Red River Waterway, Louisiana. The numerical model was used to evaluate the effects of various design changes on fine sediment deposition. These included the cross-section shape in the upstream lock approach channel, the distance between the lock wall and the first spillway gate, the number of openings in the ported guard wall, and location of a berm in the upstream channel.

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PREFACE

The numerical model investigation of the Red River upstream and downstream from Lock and Dam 3, reported herein, was conducted at the US Army Engineer Waterways Experiment Station (WES) at the request of the US Army Engineer District, Vicksburg (LMK). In addition to this numerical model study, three physical model studies of Lock and Dam 3 were conducted at WES: a fixed-bed navigation study (Report 2); a movable-bed sedimentation study (Report 3); and a hydraulic structures model study (Report 4). This is Report 5 of the series. Report 1, to be published later, will summarize all of the physical and numerical modeling studies.

The investigation was conducted during the period May 1986 to February 1988 by personnel of the Hydraulics Laboratory at WES under the direction of Mr. Frank A. Herrmann, Jr., Chief of the Hydraulics Laboratory; Richard A. Sager, Assistant Chief of the Hydraulics Laboratory; Mr. Marden B. Boyd, Chief of the Waterways Division (WD), Hydraulics Laboratory; and Mr. Michael J. Trawle, Chief of the Math Modeling Branch (MMB), WD. Mr. William A. Thomas, WD, the WES program coordinator for Red River studies, provided general guidance and review, and was a coauthor of this report. The Project Engineers and authors of this report were Mr. Ronald R. Copeland and Mr. Bradley M. Comes, MMB. Technical assistance was provided by Ms. Brenda L. Martin, MMB. This report was edited by Mrs. Marsha C. Gay, Information Technology Laboratory, WES.

During the course of this study, close working contact was maintained with Mr. Rick Robertson of the Engineering Division, LMK, who served as the coordinating engineer for LMK, providing required data and technical assistance. During this investigation, many representatives from both engineering staffs attended several meetings at WES and LMK to discuss progress of this investigation and others related to the Red River Waterway.

Commander and Director of WES during preparation of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

LIST OF FIGURES

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (force)-second per square foot	47.88026	pascals-second

RED RIVER WATERWAY, LOCK AND DAM NO. 3

SEDIMENTATION IN LOCK APPROACHES

TABS-2 Numerical Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Red River Waterway Project will provide a navigation route from the Mississippi River at its junction with Old River via the Old and Red rivers to Shreveport, LA. The project will provide a navigation channel 236 miles* long, 9 ft deep, and 200 ft wide and will include a system of five locks and dams to control water levels. Locations of the project's lock and dams are shown in Figure 1. The existing river will be realigned as necessary to develop an efficient channel, and bank stabilization and training works will be constructed to hold the newly developed channel in position.

2. Lock and Dam 3 is located at 1967 river mile 141, which is about 54 river miles upstream from John H. Overton Lock and Dam (Lock and Dam 2). The principal features at the structure are a single navigation lock, a gated spillway, and a 315-ft-long overflow weir (Figure 2). The lock chamber is located on the left descending side of the structure, has a usable length of 685 ft, and is 84 ft wide. Upstream and downstream miter gate sill elevations are 70.0** and 46.0, respectively. The lock chamber floor is at el 44.0. The lift varies up to a maximum of 31 ft. The gated spillway contains six 60-ft-wide tainter gates mounted between 9-ft-wide piers. The gate sill has an elevation of 55.0. The overflow weir is on the right descending side of the structure and has a sill elevation of 97.0.

3. The excavated channel upstream from the lock and dam is about 600 ft wide and has a design invert elevation of 64.0. In the original design, a 256-ft-wide berm at el 73.0 was located on the left descending side of the channel. The berm transitioned down to el 64.0 at the upstream end of the

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

** All elevations (el) and stages cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

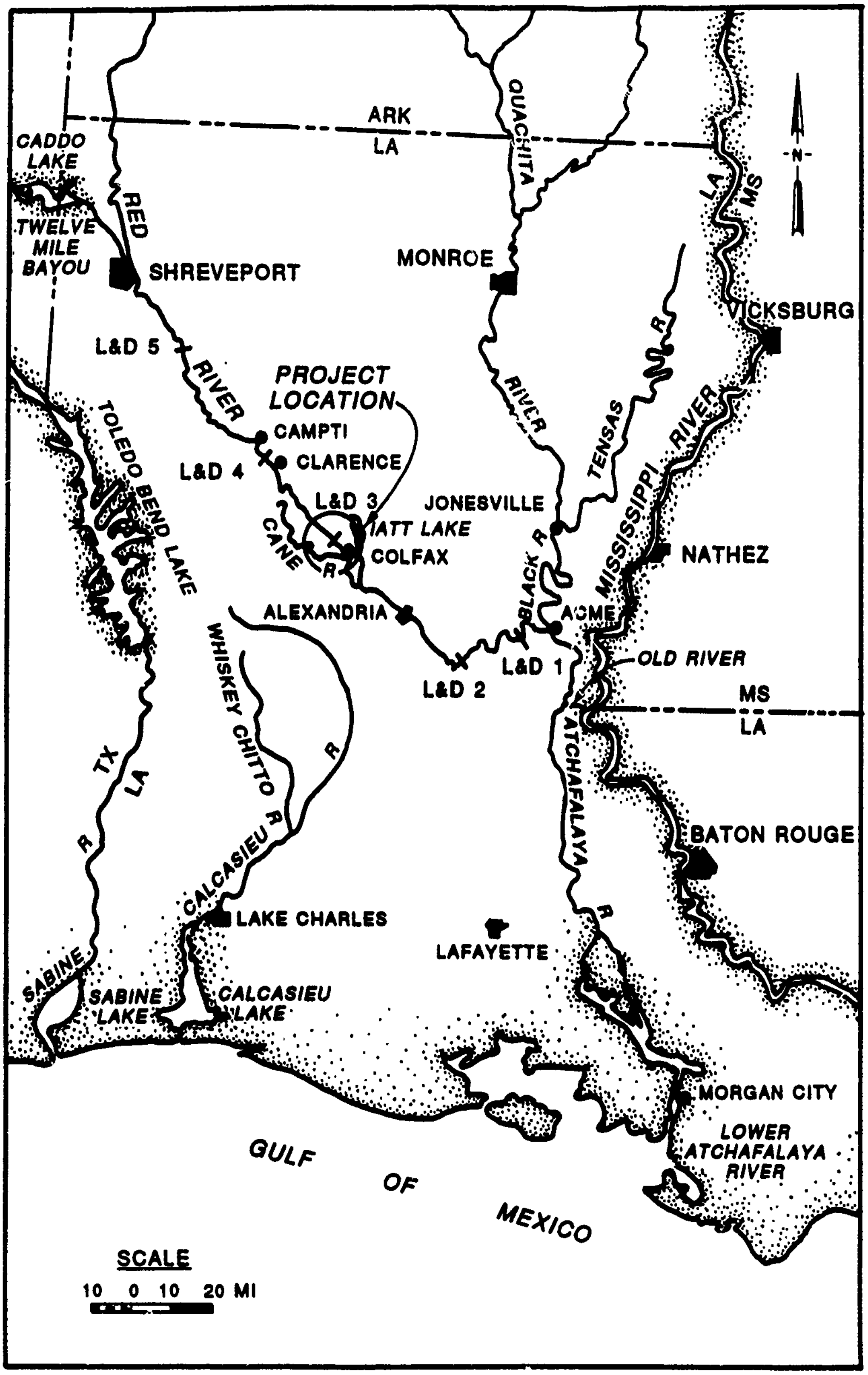


Figure 1. Location map

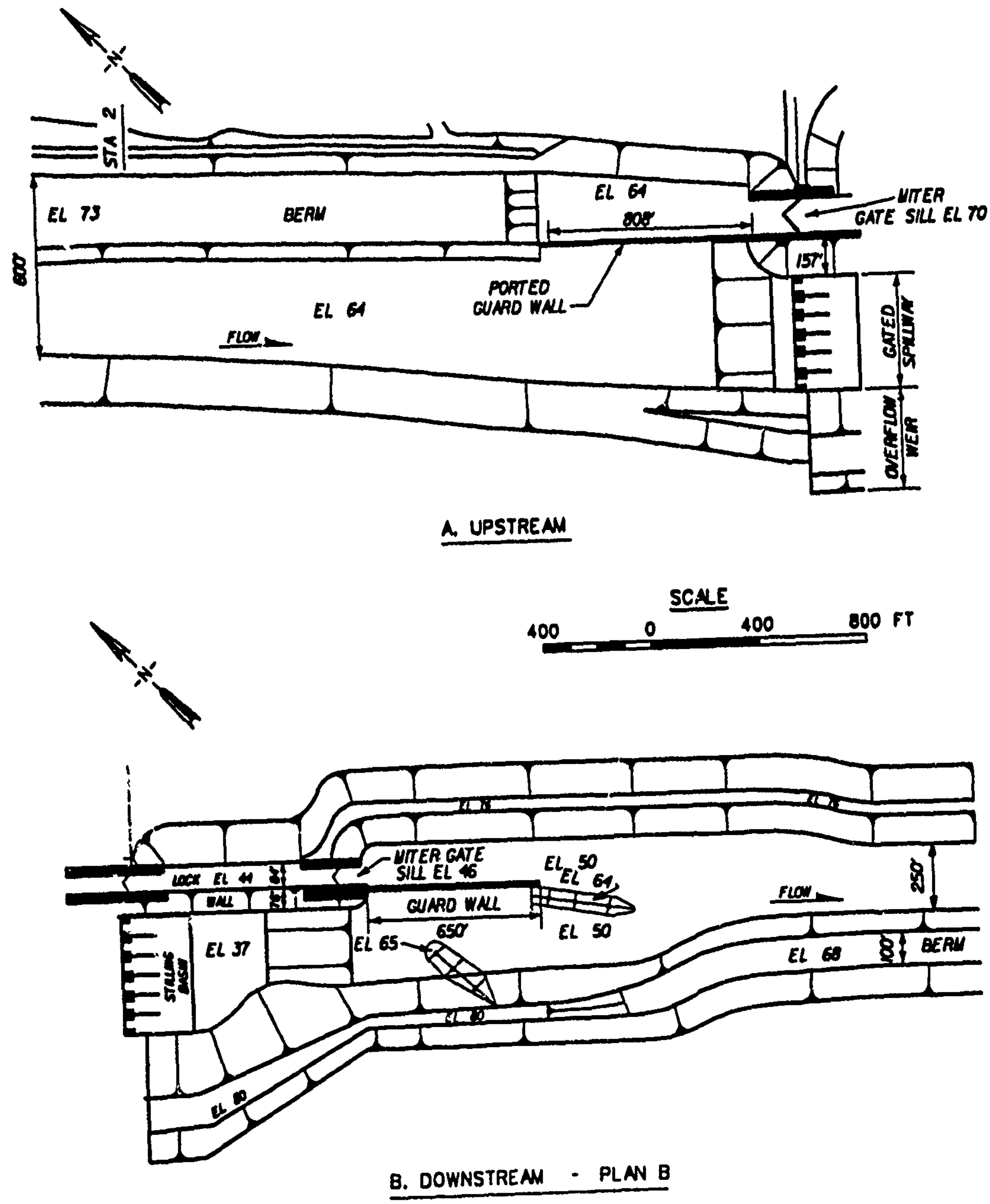


Figure 2. Project features, original design

ported guard wall. Later in the study, this berm was moved to the right descending side of the channel based on conclusions from physical model studies (Wooley, in preparation). The upstream lock approach channel is separated from the spillway entrance channel by an 808-ft-long ported guard wall. The intake manifolds for the filling system are located in a 170-ft-long lock approach section between the downstream end of the guard wall and the miter gates.

4. The excavated downstream channel has a base width of 250 ft with a design invert at el 50.0. The exit channel has a 100-ft-wide berm on the right descending side of the channel. A 650-ft-long nonovertopping guard wall separates the downstream lock approach channel from the spillway exit channel. This separation is extended by a 340-ft-long dike with a crest el of 64.0. This dike was designed to overtop at high flow, allowing water and suspended sediment into the lock approach channel while retaining bed load in the spillway exit channel. In movable-bed physical model studies conducted at the US Army Engineer Waterways Experiment Station (WES) (O'Neal, in preparation), this design reduced shoaling at the junction of the lock approach and spillway exit channels. An upstream angled dike with a sloping crest from el 80.0, land end, to el 65.0, river end, is located in the spillway exit channel on the right descending bank. This dike was designed using the movable-bed physical model to facilitate the movement of bed load.

5. The dam was designed to maintain a normal pool elevation of 95.0 and to pass all flows up to the levee design flood. The minimum downstream tailwater is el 64.0, which is the normal pool elevation maintained at Lock and Dam 2.

Purpose and Scope of the Model Study

6. Lock and Dam 1 on the Red River was opened in the fall of 1984. Deposition of fine sediment in the upstream and downstream lock approach channels was much greater than anticipated. Dredging was required at the entrance to the upstream approach channel and throughout the downstream approach channel. Sediment deposition at the downstream miter gate was severe enough to prevent operations. The lock chamber eventually had to be dewatered and the deposited sediment cleaned out. Two-dimensional numerical model studies were employed by the US Army Engineer District, Vicksburg, and WES to address the

fine sediment problem at Lock and Dam 1 (Little 1985; Copeland and Thomas 1988). As a result of these studies, design modifications were recommended and constructed at Lock and Dam 1. After almost 7 years of operation, these modifications appear to have significantly reduced the fine sediment problems in the lock approach channels.

7. The same two-dimensional numerical approach was later employed to identify possible fine sediment deposition problems prior to construction at Lock and Dam 2 (Comes, Copeland, and Thomas 1989). Lock and Dam 2, opened in the fall of 1987, experienced fine sediment deposition problems at both the upstream and downstream miter gates. However, these problems had been anticipated based on the numerical model study results. Mechanical agitating equipment, added to the lock approach channel after the structure had been placed in operation, helped keep the fine sediment from depositing at the miter gates. Based on prototype experience, improvements to the agitating system were required. The study reported herein was conducted to identify potential fine sediment deposition problems at Lock and Dam 3, using the same two-dimensional numerical modeling approach employed previously at Locks and Dams 1 and 2.

8. Separate numerical models were developed for the upstream and downstream approaches to the dam. The downstream model extended from the spillway for about one mile downstream to the end of the excavated exit channel. The upstream model extended about one mile upstream from the dam to the end of the excavated entrance channel. The primary areas of interest, in terms of fine sediment deposition, were in the lock approach channels and near the miter gates. The effects of cross-section shape in the upstream lock approach channel, the distance between the lock wall and first spillway gate, the number of openings in the ported guard wall, and the location of the berm in the upstream channel were evaluated. The Lock and Dam 3 design was also compared to the designs at Locks and Dams 1 and 2 by comparing calculated flow fields and deposition from the three studies.

9. Results of the numerical modeling of fine sediment deposition were coordinated with physical model studies conducted at WES to achieve a recommended design that adequately satisfied the needs of navigation, bed-load sediment transport, and considerations related to the hydraulic structure itself. The results of the fixed-bed navigation alignment study are given in Report 2 (Wooley, in preparation); the results of the distorted movable-bed

model study, in Report 3 (O'Neal, in preparation); and the results of the 1:50-scale structures model, in Report 4 (Maynord 1991).

PART II: THE MODEL

Description

10. The two-dimensional numerical model study was conducted using the TABS-2 modeling system (Thomas and McAnally 1985). This system provides two-dimensional solutions to open-channel and sediment transport problems using finite element techniques. It consists of more than 40 computer programs to perform modeling and related tasks. A two-dimensional depth-averaged hydrodynamic numerical model, RMA-2V, was used to generate the current patterns. The current patterns were then coupled with the sediment properties of the river and used as input to a two-dimensional sedimentation model, STUDH. The other programs in the system perform digitizing, mesh generation, data management, graphical display, output analysis, and model interfacing tasks. Although TABS-2 may be used to model unsteady flow, only steady-state conditions were simulated in this study. Input data requirements for the hydrodynamic model, RMA-2V, include channel geometry, Manning's roughness coefficients, turbulent exchange coefficients, and boundary flow conditions. The sediment model, STUDH, requires hydraulic parameters from RMA-2V, sediment characteristics, inflow concentrations, and sediment diffusion coefficients. Sediment is represented by a single grain size, and transport potential is calculated using the Ackers-White equation (Ackers and White 1973). Due to the uncertainty related to the turbulent exchange and diffusion coefficients in the two models, prototype and/or physical model data for adjustment purposes are highly desirable.

Finite Element Network

11. Finite element networks were developed to simulate about one mile of the Red River upstream and downstream from Lock and Dam 3 (Figures 3 and 4). The downstream network contained 1,419 elements and included the lock approach and spillway exit channels, dikes, and the excavated exit channel. The upstream network contained 901 elements and included the lock and spillway approach channels, the ported guard wall, and the excavated entrance channel. Conveyance through the ported guard wall was simulated in a depth-averaged sense by treating the wall as a weir, adjusting the crest elevation to achieve

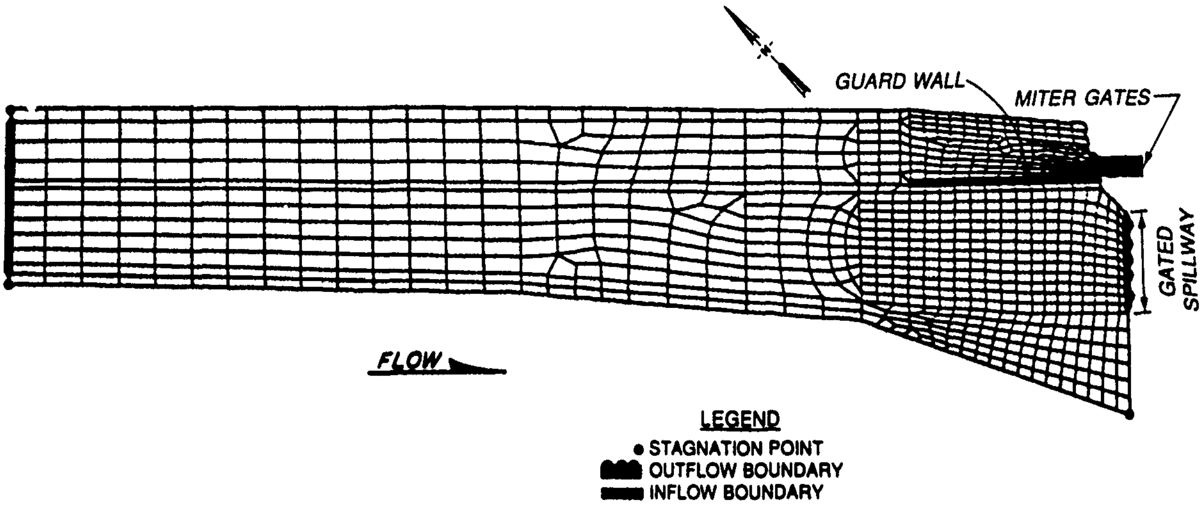


Figure 3. Finite element grid upstream from Lock and Dam 3

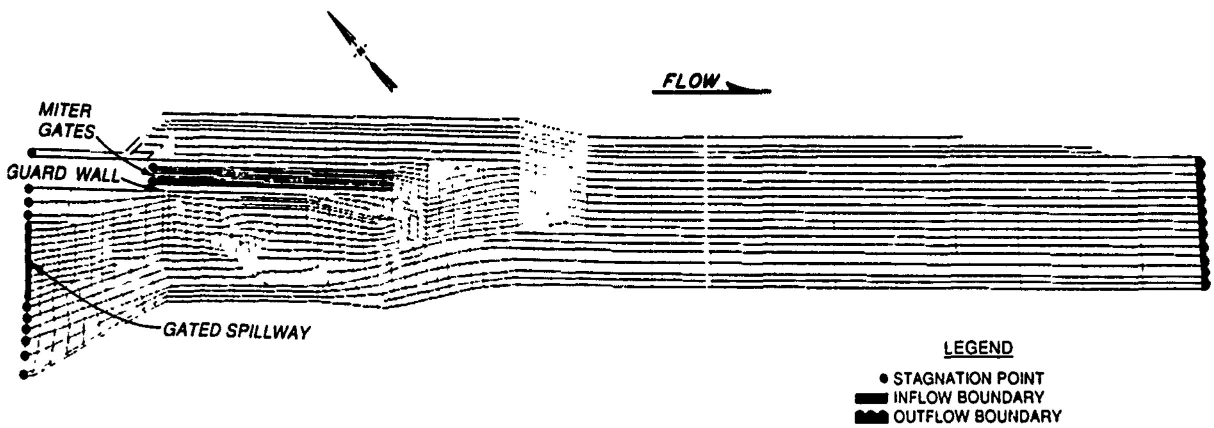


Figure 4. Finite element grid downstream from Lock and Dam 3 (Plan G)

the correct flow area, and increasing Manning's roughness coefficients to account for pier losses. Grid resolution behind the guard wall was increased to allow the model to reproduce eddies observed in the physical models. Initial bed elevations for both models were obtained from construction drawings or drawings of the navigation physical model and represent conditions prior to opening of the structure. Slip boundaries were specified for most of the grid perimeter, allowing velocities to be calculated at these locations and eliminating the need for fine grid resolution adjacent to the boundary where the lateral velocity gradient is steep. Some of the boundary nodes were specified as "stagnation points," i.e., locations of zero velocity. These specifications are generally located in corners of the grid or along boundaries with negligible flow velocities and are employed to ease calculation of grid slopes for slip boundaries. Tailwater elevation was assigned at the downstream boundary of each grid and inflow distribution specified at the upstream boundary. Grid boundary specifications are shown in Figures 3 and 4.

Hydrodynamic Boundary Conditions

12. Steady-state discharges of 80,000, 90,000, and 145,000 cfs were simulated in this study. At these discharges, the entire flow is carried through the spillway gates. The 80,000-cfs flow rate was used because, at this discharge, the upstream pool elevation is at the maximum drawdown condition on the hinged pool rating curve. The 90,000-cfs flow was used because at other locks and dams on the Red River, the spillway gates are raised and open river conditions prevail when the discharge reaches 90,000 cfs. This flow rate was used for comparison in this study. The 145,000-cfs flow rate was also used in other Red River studies and represents about a 10-year frequency flood.

13. Inflow distributions at the upstream model boundary were based on velocity measurements from the navigation physical model (Wooley, in preparation). Inflow for the downstream model was initially based on an assumption of uniform flow distribution. Later, a calculated outflow distribution from the upstream model became available and was used as the upstream boundary condition for the downstream model.

14. Tailwater elevations for the upstream model and for Plan G (discussed in paragraph 37) in the downstream model were based on rating curves at

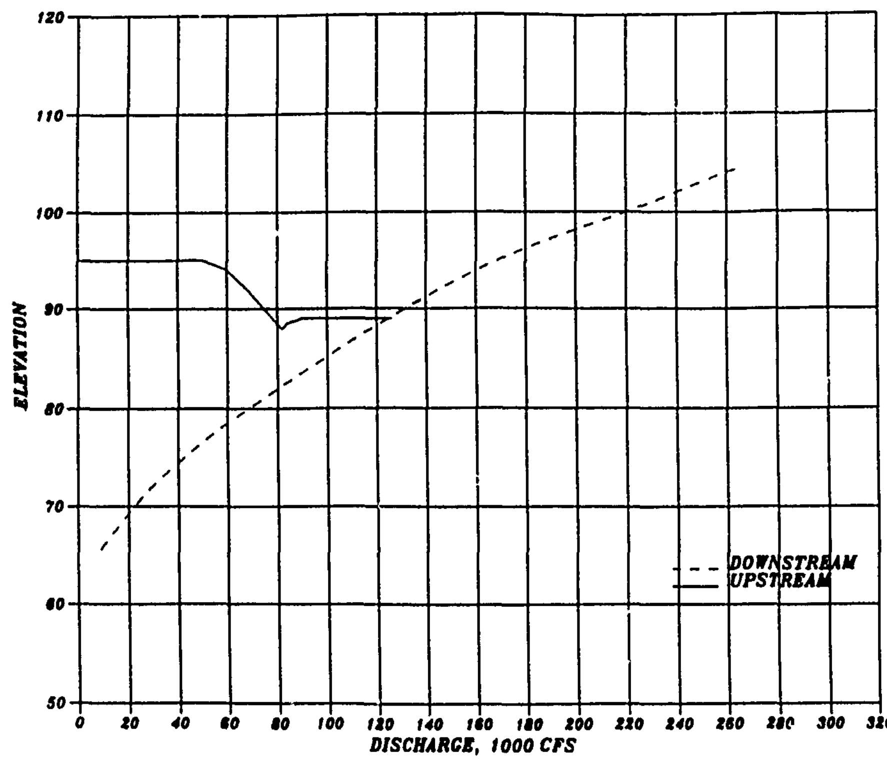


Figure 5. Tailwater rating curves

the dam provided by the Vicksburg District (Figure 5). The tailwater for Plan G, in the downstream model, tested at 80,000 cfs, was increased from el 82, from the rating curve, to el 83.7 as a result of reanalysis of the original backwater calculations by the Vicksburg District. During the initial phase of the study, when Plan B (described in paragraph 34) was being tested in the downstream model, a rating curve slightly different from that shown in Figure 5 was used. Tailwaters for Plan B tests are shown in the following tabulation.

Model	Discharge cfs	Water-Surface Elevation
Upstream	80,000	88.0
	145,000	92.2
Downstream	80,000	83.7*
	90,000	83.6**
	145,000	92.2*
	145,000	93.7**

* Plan G
** Plan B

the tailwater for the downstream model was designated at a location approximately 500 ft downstream from the end of the guard wall, centered in the spillway exit channel. The tailwater for the upstream model was taken to be in the center of the spillway approach channel at the end of the ported guard wall. The downstream boundaries of the numerical models were adjusted until the elevation at the designated tailwater locations matched the elevation from the rating curves. Normal pool elevation of the dam is 95 ft; however, when flow exceeds 50,000 cfs, a hinged pool operation lowers the elevation upstream from the dam. The maximum drawdown condition on the hinged pool rating curve occurs at el 88 and a discharge of 80,000 cfs. Designated downstream boundary elevations for the discharges tested are listed in the tabulation.

Roughness Coefficients

15. Manning's roughness coefficients were assigned to each element. The roughness coefficient for elements on the channel bottom with a sand bed was set at 0.017. This value, used by Vicksburg District in their study upstream of Lock and Dam 1 (Little 1985), is based on grain size and water-surface elevation adjustments to their numerical model. Riprap placed on the channel bottom, side slopes, and dikes had a median particle size D_{50} that varied between 9 and 36 in. The Limerinos equation (Limerinos 1970), which includes relative roughness as a variable, was used to calculate roughness coefficients for the different features through a range of depth:

$$n = \frac{0.0926R^{0.1667}}{1.16 + 2.0 \log \left(\frac{R}{D_{84}} \right)} \tag{1}$$

where

n = Manning's roughness coefficient

R = hydraulic radius, ft

D_{84} = particle size of which 84 percent of the bed is finer, ft

Average depths were used to calculate a roughness coefficient on side slopes. Calculated values were adjusted slightly to account for additional losses due to disturbance of the hydrostatic velocity distribution. The following roughness coefficients were assigned in the numerical model:

Feature	Value
Sand bottom	0.017
Riprap bottom	0.040
Riprap side slope	0.045
Boundary elements	0.055
Riprap berm	0.045
Upstream guard wall	0.060
Submerged dikes	0.060

These values are compatible with those used in previous work at locks and dams on the Red River.

Turbulent Exchange Coefficients

16. Momentum exchanges due to velocity gradients are approximated in RMA-2V by multiplying a turbulent exchange coefficient times the second derivative of the velocity with respect to the x- and y-directions. Limited guidance is available for selection of these coefficients. Previous studies of Red River locks and dams (Little 1985; Copeland and Thomas 1988; Comes, Copeland, and Thomas 1989) have verified values for these coefficients using measured data from physical models as well as the prototype. Sensitivity studies conducted in these previous studies indicated that a coefficient of 25 $lb\text{-}sec/ft^2$ was satisfactory. For this study, the element aspect ratios remained approximately the same as in previous studies and a turbulent exchange coefficient of 25 $lb\text{-}sec/ft^2$ was selected.

Bed Material

17. The TABS-2 system analyzes sediment movement using a representative grain size. This technique works well with fairly uniform bed material. Unfortunately, bed material size varies considerably around a structure as well as laterally across a channel. Therefore, grain size must be representative of the area of primary interest in the river. For this study, measured data from Lock and Dam 1 were applied at Lock and Dam 3. This translocation of bed material data was required because Lock and Dam 3 was not in operation and direct data were not available. Previous one-dimensional numerical model work (Copeland and Thomas 1988) demonstrated the soundness of this translocation by

showing that the variation in average bed material gradation in the Red River between Shreveport and Old River is slight. Bed samples from deposits in the upstream and downstream lock approach channels at Lock and Dam 1, taken in April and May of 1985, had median diameters between 0.07 and 0.04 mm. The Vicksburg District chose a representative grain size of 0.07 mm for their upstream numerical model study (Little 1985). Differences between these measurements and subsequent measurements upstream and downstream from Lock and Dam 1 were deemed insufficient to forsake consistency, and an average grain size of 0.07 mm was adopted for numerical simulations of deposition in the lock approach channels for studies on the Red River.

Sediment Concentration

18. The sediment inflow concentration for the numerical model is a function of the representative grain size used in the model. Only the portion of the total sediment load that contributes to bed changes in the primary area of interest should be included. Using the bed material gradation in the upstream lock approach channel at Lock and Dam 1, a median diameter of 0.07 mm and a minimum diameter of 0.03 mm were determined. Sediment inflow concentrations to the model were based on measured concentrations of material greater than 0.03 mm.

19. The Vicksburg District obtained suspended sediment measurements upstream from Lock and Dam 1 at river mile 51.5 in April and May 1985. At a discharge of 59,500 cfs, a total suspended sediment concentration of 771 mg/ℓ was measured, and at 93,000 cfs the measured concentration was 1,525 mg/ℓ. These concentrations are compared to concentrations at Alexandria, LA (river mile 104) for the period 1971, 1972, and 1975-1981 in Figure 6. The 1985 measured concentrations are within the range of data, but well below average values. This distribution may have occurred because the 1985 data were taken well into the runoff season when concentrations typically decline. Fifty-three percent of the total measured suspended load was greater than 0.03 mm. This percentage was used to determine the sediment inflow concentrations for the numerical model.

20. Extrapolation and interpolation of the two 1985 data points were used to determine sediment inflow concentrations of 720, 850, and 1,750 mg/ℓ for discharges of 80,000, 90,000, and 145,000 cfs, respectively.

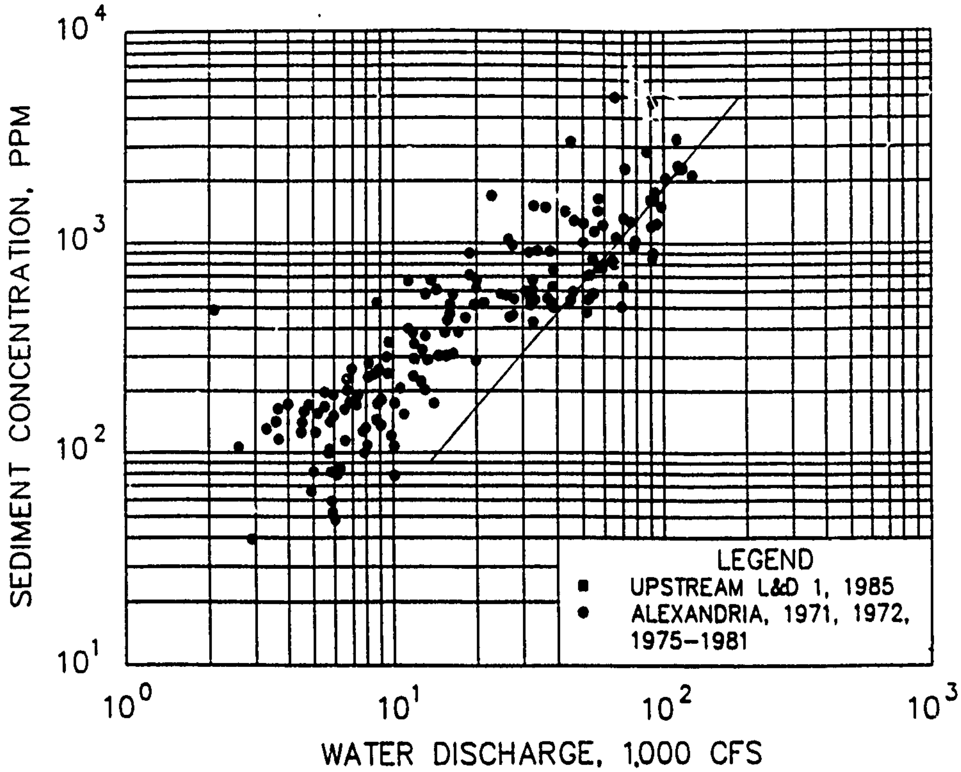


Figure 6. Inflow sediment concentrations

Sediment Diffusion Coefficients

21. The same sediment diffusion coefficients used in previous numerical model studies of locks and dams on the Red River (Little 1985; Copeland and Thomas 1988; Comes, Copeland, and Thomas 1989) were used in this study. Sensitivity studies conducted as part of those studies indicated that calculated deposition was not sensitive to the sediment diffusion coefficient in areas where conveyance is the primary driving force affecting sediment movement. However, in essentially dead-water areas, such as in front of the lock miter gates, where deposition is primarily a function of diffusion, the sediment diffusion coefficients are critical. Reproduction of hydrographic survey data from the upstream and downstream lock approach channels at Lock and Dam 1 was used to determine the appropriate coefficients for these models.

22. Deposition in the downstream lock approach channel at Lock and Dam 1 measured between October 1984 and May 1985 was compared to calculated deposition using a sediment diffusion coefficient of 2 m^2/sec (Figure 7). This simulation was especially good for the first 500 ft downstream from the

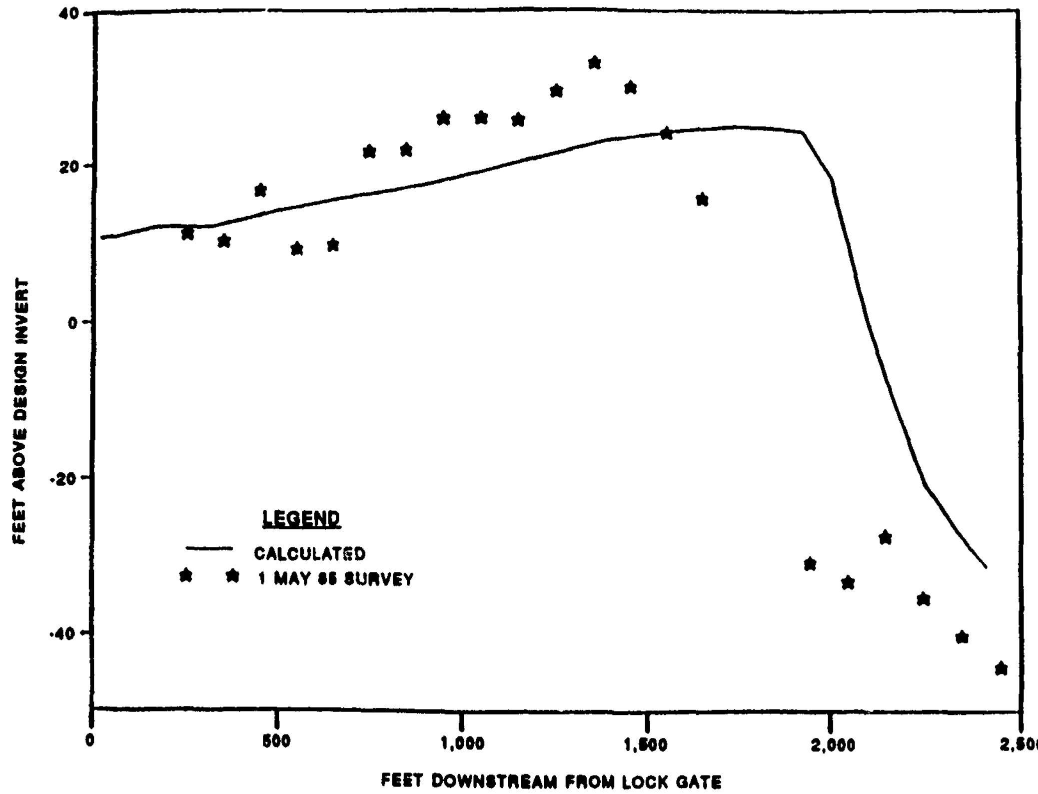


Figure 7. Comparison of measured and calculated deposition at Lock and Dam 1, downstream lock approach channel

lock gate. For the next 1,000 ft, the model predicted about 75 percent of the measured deposition.

23. Deposition in the upstream lock approach channel at Lock and Dam 1 measured between 4 December 1985 and 17 December 1985 was compared to calculated deposition using sediment diffusion coefficients of 0.5, 2.0, and 25.0 m^2/sec. The primary function of the sediment diffusion coefficients is to move fine sediments into the dead-water zones. The prototype measurements indicated that the material moved approximately 500 ft into the dead-water zone. Sediment diffusion coefficients of 0.5, 2.0, and 25.0 m^2/sec moved material 50, 600, and 1,200 ft, respectively, into the dead-water zone.

24. The Lock and Dam 1 upstream and downstream numerical models were deemed to have successfully reproduced the prototype data in the primary area of interest using a sediment diffusion coefficient of 2.0 m^2/sec. This value was used on all numerical model studies of the Red River because hydraulic conditions and model grid resolution were similar.

PART III: MODEL RESULTS

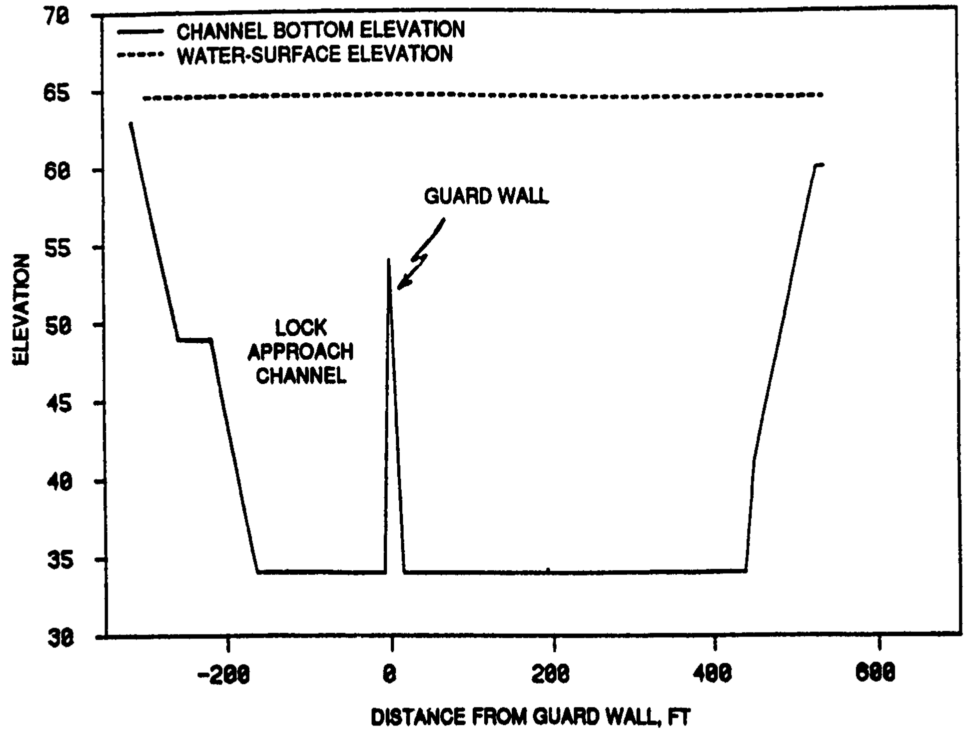
Original Upstream Design

25. The distinguishing features of the original upstream design included an 808-ft-long ported guard wall with fourteen 42-ft-wide ports and one 21-ft-wide port at the downstream end of the guard wall; a berm on the left descending bank, upstream from the ported guard wall; and a 157-ft separation between the inside lock chamber wall and the first spillway gate. Calculated velocity vectors in the downstream portion of the upstream model for a steady-state discharge of 80,000 cfs are shown in Plate 1. Plate 1 indicated an eddy behind the dam's overflow weir as well as one near the miter gates.

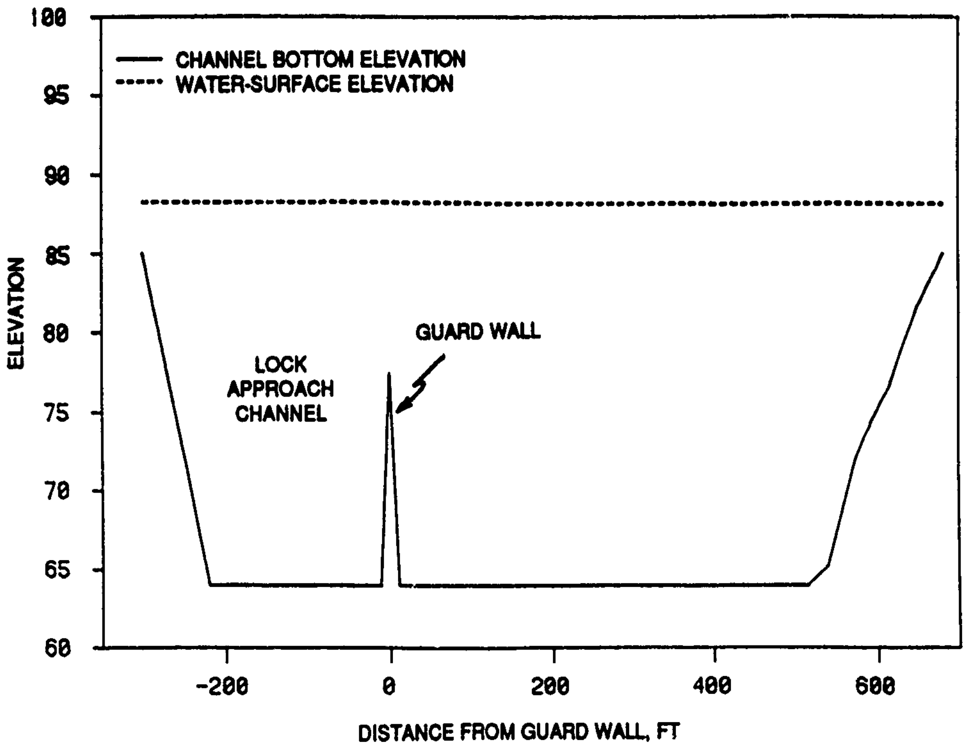
26. To obtain a qualitative evaluation of deposition potential in the lock approach channel, an arbitrary 10-day simulation of 80,000 cfs with a sediment inflow concentration of 720 mg/ℓ was conducted with the numerical model. During the 10-day simulation, about 2 ft of fine sediment deposited near the miter gates and a large deposit formed on the left bank of the lock approach channel (Plate 2). This deposit was greater than that predicted at Lock and Dam 2 after a 10-day simulation at 90,000 cfs and a sediment inflow concentration of 850 mg/ℓ (Plate 2).

27. Differences in river cross sections at the upstream end of the lock approach channels of Locks and Dams 2 and 3 are shown in Figure 8. The total cross-sectional area at Lock and Dam 2 was about 10 percent larger than at Lock and Dam 3; the bottom width was about 20 percent narrower, but the depth was about 20 percent greater.

28. The lock approach channel's cross sections at Locks and Dams 2 and 3 are compared in Figures 9 and 10. Figure 9 shows cross sections at the upstream end of the guard wall, and Figure 10 shows cross sections near the downstream end of the wall. The channel bottom width at the upstream cross section was about 25 percent narrower at Lock and Dam 2. The bottom widths at the downstream section was about 15 percent wider at Lock and Dam 2. Calculations at both Locks and Dams 2 and 3 indicated that approximately 16 percent of the total discharge flowed through the upstream cross section. At Lock and Dam 3, 8 percent of the total discharge passed the downstream cross section, compared to 14 percent at Lock and Dam 2. The angled bank line at Lock and

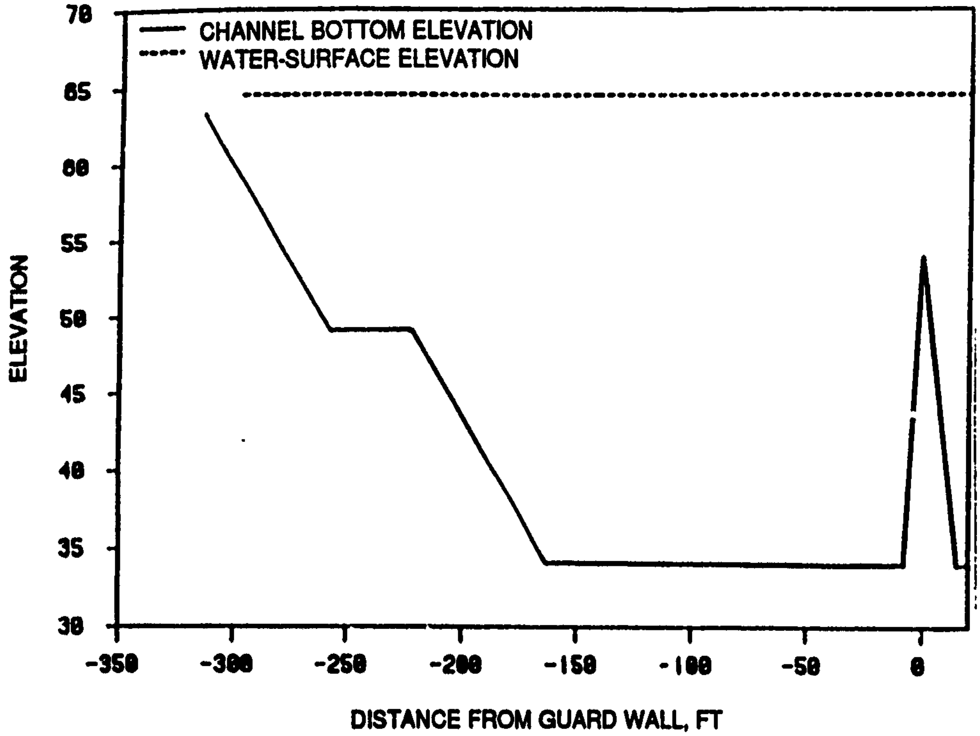


a. Lock and Dam 2, discharge 90,000 cfs

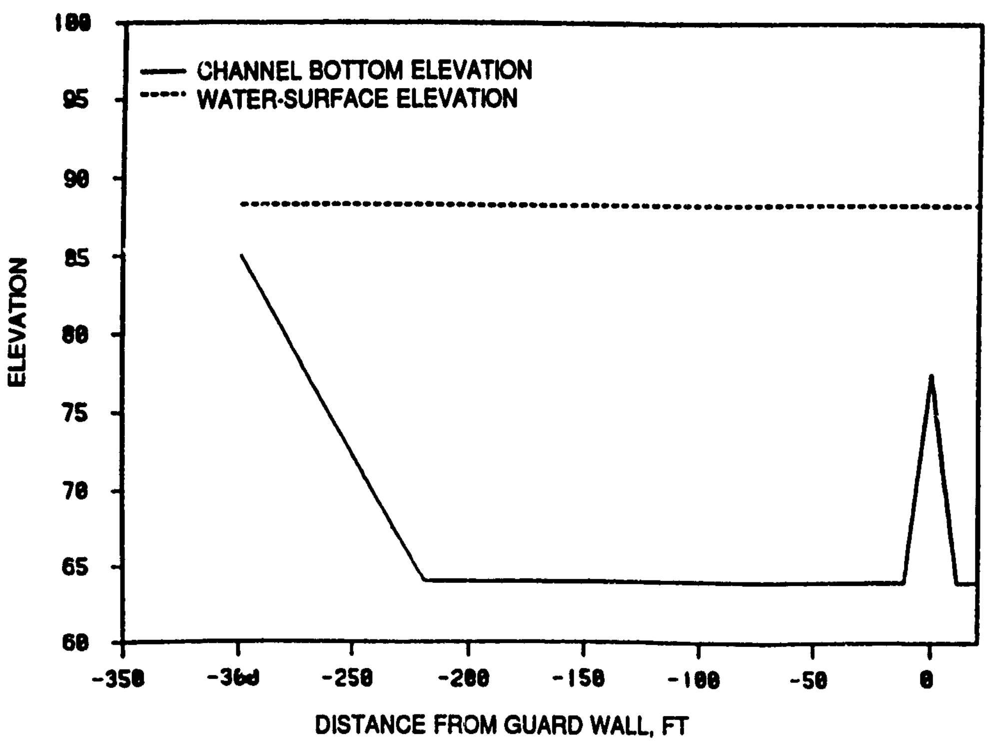


b. Lock and Dam 3, discharge 80,000 cfs

Figure 8. Comparision of channel cross sections at upstream end of guard wall

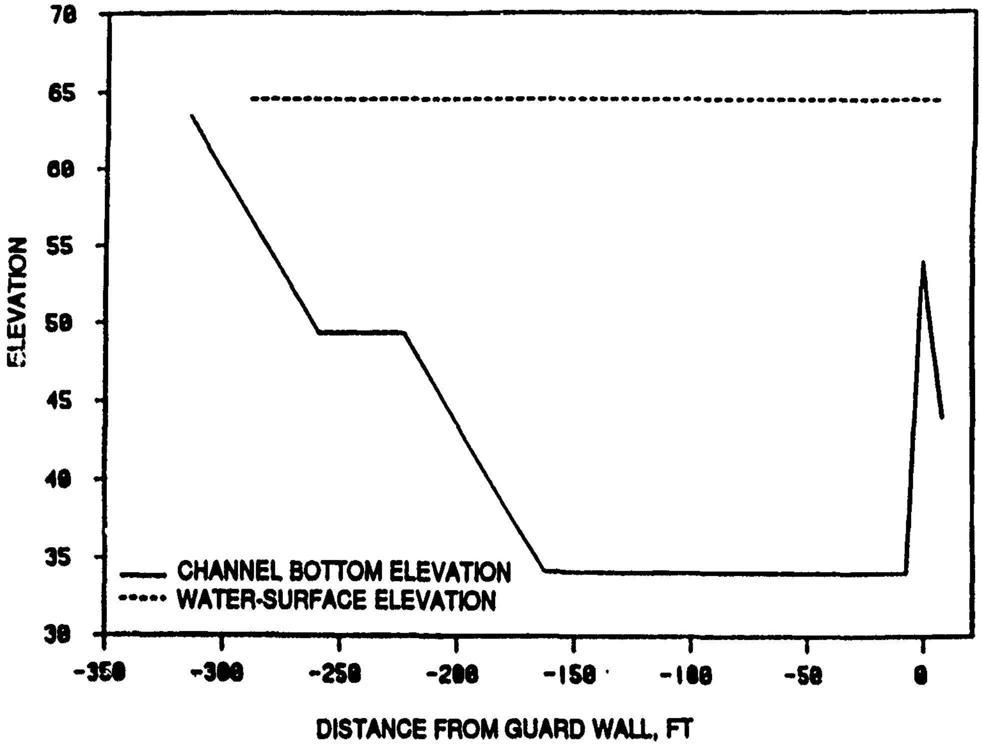


a. Lock and Dam 2, discharge 90,000 cfs

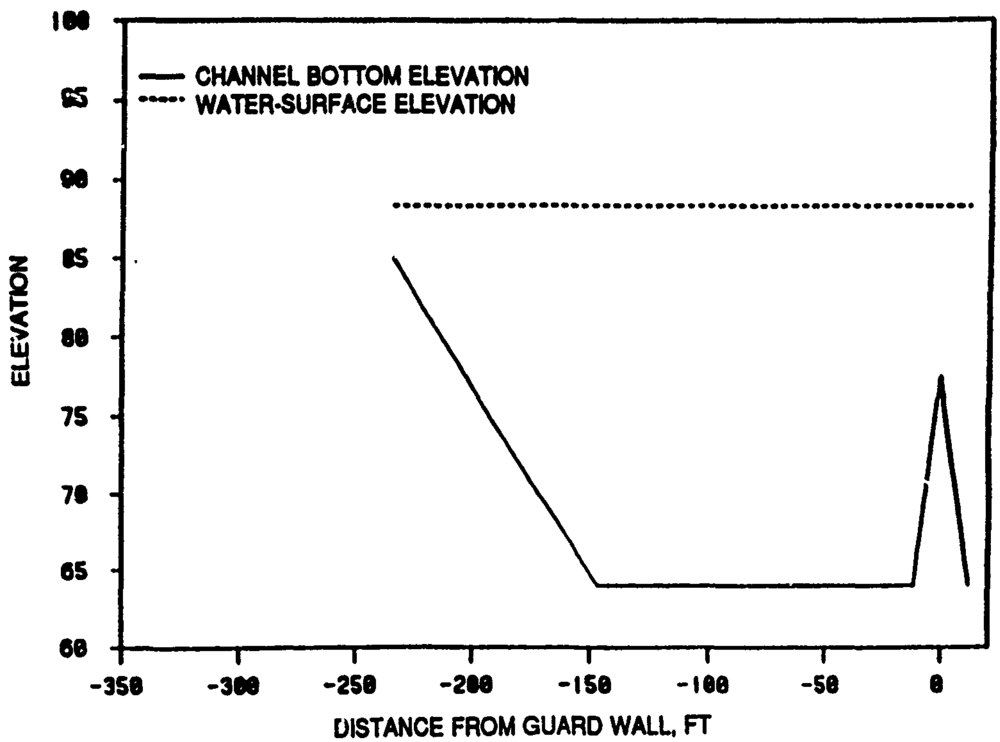


b. Lock and Dam 3, discharge 80,000 cfs

Figure 9. Lock approach channel cross sections, upstream end of guard wall



a. Lock and Dam 2, discharge 90,000 cfs



b. Lock and Dam 3, discharge 80,000 cfs

Figure 10. Lock approach channel cross sections, downstream end of guard wall

Dam 3 may direct flow toward the guard wall. The calculated velocity vectors for each case are shown in Plate 3. Notice that the angle of attack relative to the guard wall was greater at Lock and Dam 3 than at Lock and Dam 2.

29. Velocity magnitudes and bed shear stresses were also compared to calculated results from the Lock and Dam 2 model. Plate 4 shows velocity magnitude contours at Locks and Dams 2 and 3. The steep gradient located at the upstream end of the guard wall is due to flow off the end of the berm. Velocity contours at Lock and Dam 2 (Plate 4) indicated that the same velocity contours were oriented parallel to and located near the guard wall. Comparing Plates 2 and 4, it can be seen that deposition occurred where velocities were less than 1 fps at Lock and Dam 2, while at Lock and Dam 3, deposition occurred in areas with velocities as high as 2 fps. The lack of good correlation between velocity contours and deposition initiated an investigation of the bed shear stresses. Plate 5 shows the bed shear stresses for both sites. Comparison of this plate to Plate 2 shows that the shear stress contours match the deposition patterns. At both Locks and Dams 2 and 3, deposition occurred when bed shear stresses were less than 0.025 lb/ft^2.

30. A flow of 80,000 cfs was simulated for an additional 10-day period to see if the deposition rates changed. This was accomplished by calculating steady-state hydrodynamics with RMA-2V, and then running the sediment model, STUDH, for 10 days. A new geometry file was created which reflected the deposition that occurred during the 10-day simulation. New hydrodynamic calculations were made with RMA-2V, followed by another 10-day simulation with STUDH. The calculated deposition rate was unchanged.

Effect of Distance between Lock Wall and Spillway

31. The effect of the increased distance (relative to Lock and Dam 2) between the lock wall and the left pier of the first spillway bay at Lock and Dam 3 was tested in the upstream numerical model. The downstream boundary of the upstream numerical model was changed so that the distance between the inside of the lock wall and the left pier of the spillway bay was reduced from 157 to 76 ft, which is the same as at Lock and Dam 2. The test was conducted with a discharge of 80,000 cfs. With this change there was less deposition, but higher velocities, behind the guard wall. Velocity vectors from the original design are compared with those of the test plan in Plate 6. The

velocity vectors behind the guard wall run parallel to the wall and show a larger magnitude in the test plan. Conveyance was determined at two locations behind the guard wall: location 1 was approximately 100 ft upstream from the upstream end of the guard wall, and location 2 was approximately 450 ft downstream from the upstream end of the guard wall. Computations at location 1 were along a line extending from the center line of the guard wall to the left descending bank. Computations at location 2 extended from the guard wall to the left bank. In the original design, 19.1 percent and 10.8 percent of the total flow passed locations 1 and 2, respectively. With the distance between the lock wall and spillway gates reduced, 21 and 17.8 percent passed locations 1 and 2, respectively. Notice also that the outflow distribution through the spillway shifted from right-skewed for the original design to left-skewed for the test plan. The calculated deposition patterns for a 10-day simulation are compared with those of the original design in Plate 7. The results support the hypothesis that the additional separation between the lock and the first spillway bay contributed to the difference between the deposition patterns at Locks and Dams 2 and 3. The deposition pattern on the right descending bank in the test plan is larger due to the longer overflow weir (nonovertopping at this discharge) that resulted from moving the spillway to the left. The maximum depths of deposits near the miter gates and downstream of the berm are within one-half foot of one another; however, in the test plan the deposit near the downstream end of the guard wall does not extend across the lock approach channel as it does in the original design.

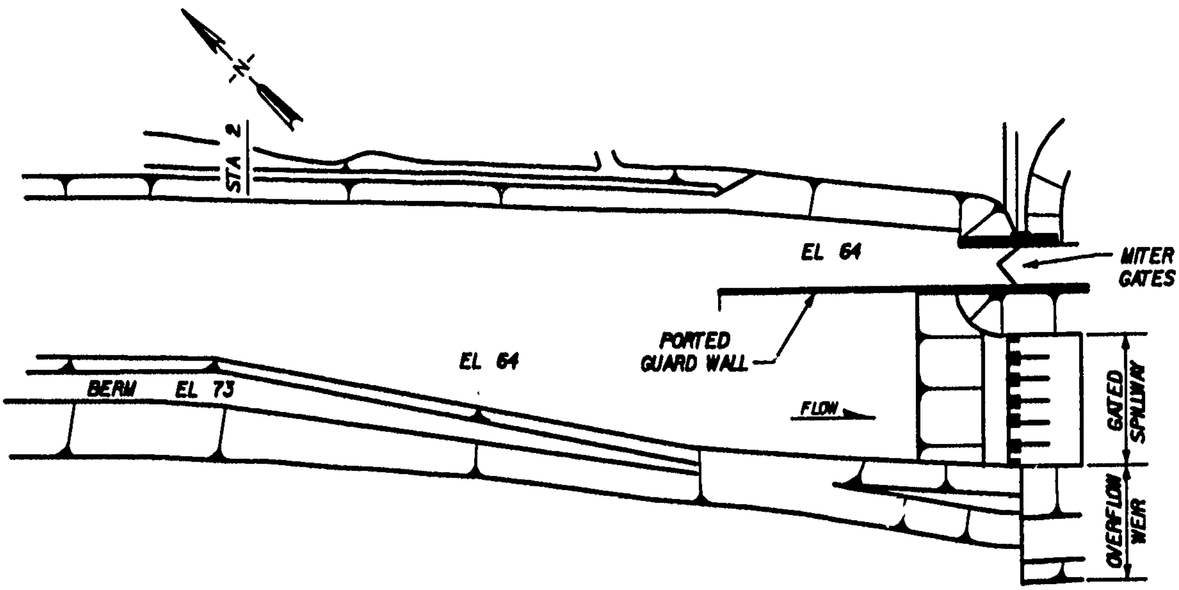
Effect of Closing Guard Wall Ports

32. The effect of closing some of the guard wall ports and thus reducing the flow in the lock approach channel was tested with the numerical model. The seven upstream ports in the guard wall were closed. Velocity vectors are compared with those of the original design in Plate 8. With this design, 17.0 percent of the total flow passed location 1 and 10.9 percent passed location 2, compared to 19.1 and 10.8 percent, respectively, in the original design. The velocity vectors were parallel with the solid guard wall, but there was an outdraft at about the same direction as in the original design on the downstream one-half of the wall plus an additional outdraft upstream from the guard wall. Plate 9 compares deposition patterns for this test and the

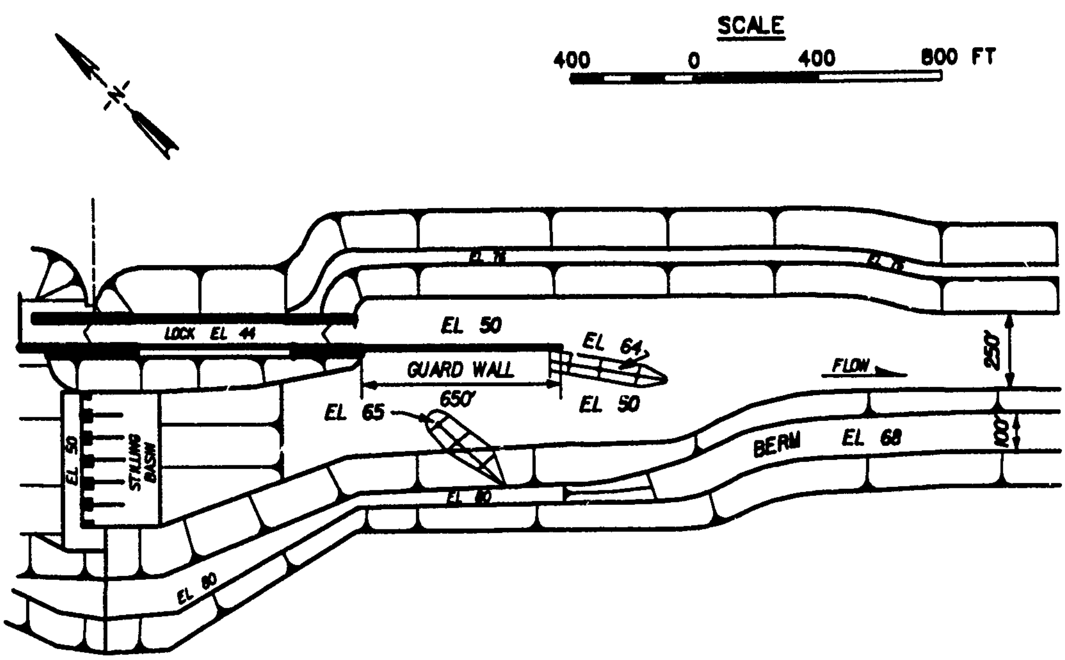
original design. A small change in shape is evident where the solid portion of the guard wall ends, but deposits continue to resemble those for the original design along the downstream end of the guard wall.

Revised Berm Location

33. In Plan G, developed in the navigation physical model (Wooley, in preparation), the berm on the left descending bank was removed and placed on the right descending bank at the same elevation (el 73) (Figure 11). The distance between the inside of the lock wall and the spillway gate remained at 157 ft. The purpose of this change was to prevent the flow from drifting away from the left descending bank, a condition unfavorable to navigation. Inflow distribution at the upstream boundary of the numerical model was based on measured velocities taken from the navigation physical model. Velocity measurements were also taken from the physical model in the lock approach channel at the upstream end of the guard wall. These measurements were then compared to calculated velocities from the numerical model at the same location. Both models indicated that approximately 25 percent of the total flow entered behind and passed through the ported guard wall. The velocity and deposition patterns for the 80,000-cfs test are shown in Plates 10 and 11, respectively. The velocity patterns show low velocities on the berm. The flow approaching the ported guard wall indicates a minimal outdraft at the upstream end and a concentration of the flow through the downstream ports. The deposition patterns for a 10-day simulation show up to 3 ft of deposition on the right overflow weir and on the left descending bank line near the upstream end of the guard wall. Approximately 2 ft of material deposited in front of the upstream miter gates. Compared to the original design, more fine sediment deposited on the right descending bank because of lower velocities over the berm, less sediment deposited behind the ported guard wall, and deposition in front of the miter gates was about the same. Plates 10 and 11 show the velocity and deposition patterns in the upstream model for the 145,000-cfs test (Plan G). This condition produced velocities of 8 fps in the lock approach channel. Flow through the ported guard wall was also heavily concentrated toward the downstream end of the wall. The deposition patterns for a 10-day simulation show approximately 9 ft of deposition on the overflow weir and in a spot near



A. UPSTREAM



B. DOWNSTREAM

Figure 11. Project features, Plan G

the upstream end of the guard wall. Approximately 7 ft of material deposited against the lock miter gates.

Original Downstream Design, Plan B

34. The initial geometry for the downstream model was based on Plan B (Figure 2), which was developed from movable-bed physical model studies at WES (O'Neal, in preparation). This plan included a 650-ft-long nonovertopping guard wall and a 350-ft-long submerged dike between the lock approach and spillway channels. Including the length of the lock chamber wall downstream from the miter gates, the total nonovertopping distance was 800 ft. A spur dike, angled upstream, was located in the spillway exit channel to facilitate the movement of bed load. Initial bed elevations in the numerical model were representative of conditions prior to opening the structure. For this initial test, inflow distributions were assumed to be uniform through the spillway gates. Calculated values from the upstream numerical model were unavailable at the time the original downstream model was tested. Steady-state discharges of 90,000 and 145,000 cfs were simulated with RMA-2V. Calculated velocity vectors for the upstream two-thirds of the model are shown in Plate 12. At both discharges, a large eddy developed in the lock approach channel near the end of the nonovertopping guard wall.

35. The sediment transport model, STUDH, was used to simulate 10-day-duration steady flows Q of 90,000 and 145,000 cfs with sediment inflow concentrations C of 850 and 1,750 mg/ℓ, respectively. Calculated deposition profiles down the center line of the lock approach channel are shown in Figure 12. Maximum deposition occurred just downstream from the end of the guard wall. The maximum 10-day deposition was 3.6 and 10.5 ft for 90,000 and 145,000 cfs, respectively. Deposition decreased toward the miter gates where 10-day deposition was 0.6 and 4.4 ft for 90,000 and 145,000 cfs, respectively.

36. The sediment transport model was used to compare the downstream design at Lock and Dam 3 to the as-built designs at Locks and Dams 1 and 2. For the comparisons, a steady-state discharge of 90,000 cfs was simulated for 10 days with a sediment inflow concentration of 670 mg/ℓ. This sediment concentration was used because it was used in the numerical model at Lock and Dam 1. Results, shown in Figure 13, indicated less deposition at the lock miter gates at Lock and Dam 3. Deposition at Lock and Dam 3 was about

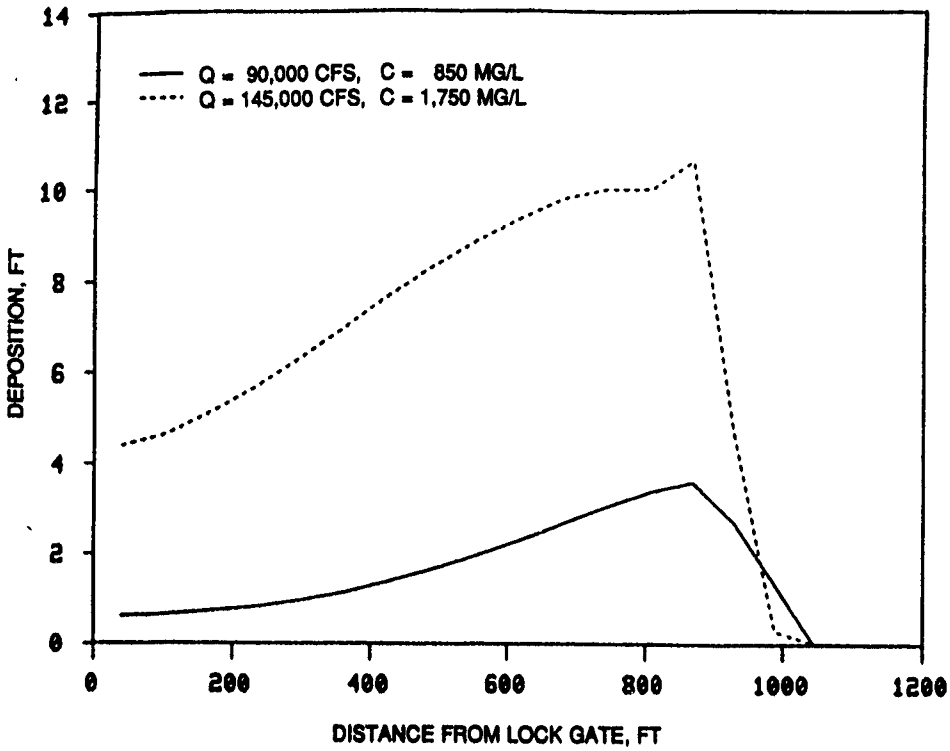


Figure 12. Calculated deposition along center line of downstream lock approach channel, Plan B

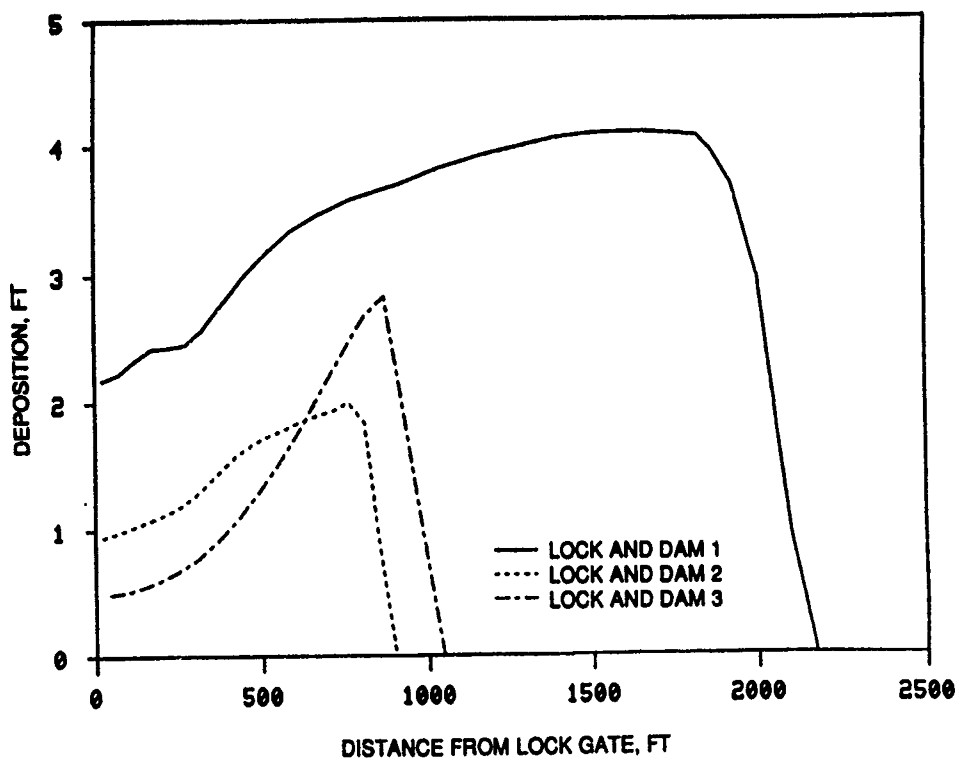


Figure 13. Comparison of deposition in downstream lock approach channel at Locks and Dams 1, 2, and 3 (Lock and Dam 3 is Plan B)

25 percent of that calculated at Lock and Dam 1. At Lock and Dam 2, deposition was about 45 percent of that calculated at Lock and Dam 1. This improvement is attributed to the increased nonovertopping distance between the miter gates and the end of the guard wall. This is consistent with numerical model and prototype experience at Lock and Dam 1, where modifications that included extending a nonovertopping wall improved conditions (Copeland, Combs, and Little 1989). The MARKER graphics program had not been developed when the original plan was tested; therefore, no deposition contour plots are presented herein.

Downstream Design, Plan G

37. In the initial downstream plan, the separation between the lock and spillway gates was 76 ft. Subsequently, this distance was increased to 157 ft, requiring modification of the right descending bank downstream from the spillway (Plan G, Figure 11). These modifications were incorporated into the model and run for steady-state discharges of 80,000 and 145,000 cfs. A discharge of 80,000 cfs was used instead of 90,000 cfs to be consistent with upstream model results, which were available when this plan was tested. In these tests, inflow boundary conditions were based on calculated outflow from the upstream numerical model. Sediment inflow concentrations of 720 and 1,750 mg/ℓ were used.

38. Velocity vectors and deposition patterns from the two tests are shown in Plates 13 and 14, respectively. The velocity patterns indicate the presence of two small eddies, one on each side of the spillway gates, and a large eddy downstream and to the left of the guard wall. The flow through the spillway gates expanded and attacked the right descending bank, which was contracting. This expansion and contraction causes nonuniform flow lines to extend downstream until the flow passes over the submerged dike on the right descending bank. During a 10-day, 80,000-cfs simulation, a maximum of about 4 ft of material deposited near the spillway gates and approximately 3.5 ft deposited just downstream from the end of the guard wall (Plate 14). The deposit from the large eddy extended upstream toward the miter gates to approximately the midpoint of the guard wall. During a 10-day, 145,000-cfs simulation, the maximum depth of deposit near the spillway gates was 10 ft. The deposit near the guard wall approached 9 ft and extended upstream to the

miter gates where a 2-ft deposit formed against the downstream miter gates. Deposition profiles for 80,000 and 145,000 cfs along the lock approach center line are shown in Figure 14. The results from Plans B and G are not directly comparable due to differences in discharges tested, tailwaters, and inflow distribution.

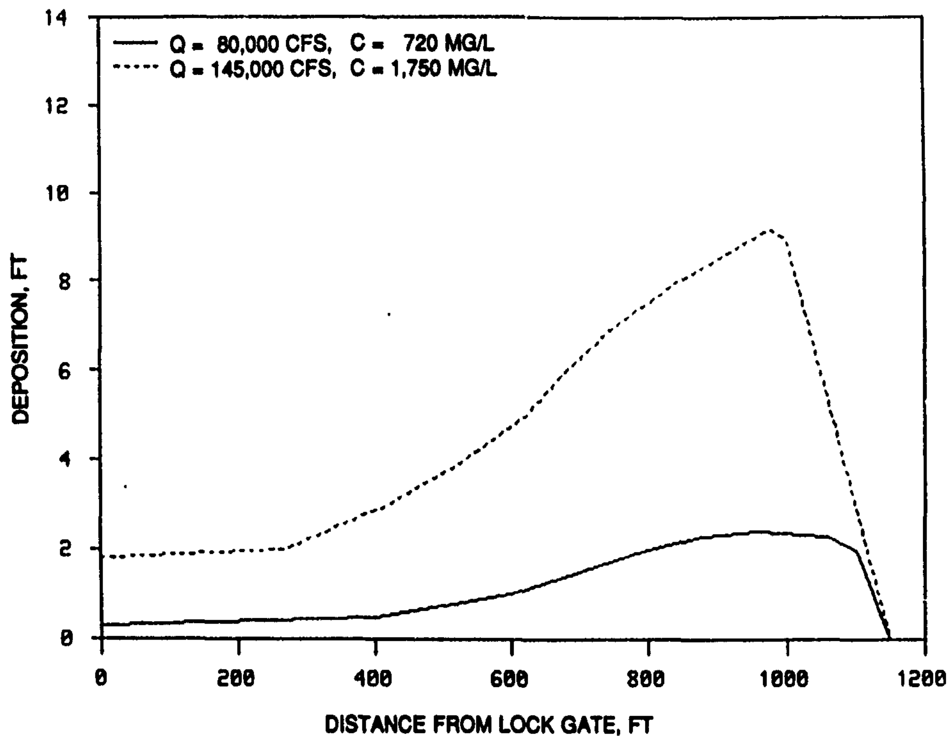


Figure 14. Calculated deposition along center line of downstream lock approach channel, Plan G

PART IV: CONCLUSIONS

39. The two-dimensional numerical model study demonstrated that there will be fine sediment deposition problems in front of the upstream and downstream miter gates and in the lock approach channels at Lock and Dam 3. Calculated deposition for a 10-day simulation at the upstream miter gates was about 2 ft for a steady-state discharge of 80,000 cfs and about 7 ft for a steady-state discharge of 145,000 cfs. Downstream, deposition for a 10-day simulation was less than 0.5 ft and 2 ft for steady-state discharges of 80,000 and 145,000 cfs, respectively. These problems are of the same order of magnitude as were calculated at Lock and Dam 2, where mechanical removal contingencies have been required to maintain operability of the lock.

40. Reducing the distance between the lock wall and the spillway gate reduced deposition in the upstream lock approach channel, but not at the miter gate.

41. Closing off seven ports at the upstream end of the ported guard wall did not significantly affect fine sediment deposition in the lock approach channel nor at the miter gate.

42. Moving the berm in the upstream excavated channel from the left descending bank to the right descending bank increased flow behind the guard wall in the 80,000-cfs test. This increase amounted to about 4 percent of the total riverflow, i.e., from 21 to 25 percent of the total riverflow. At 80,000 cfs, more fine sediment deposited on the right descending bank due to the lower velocities over the berm. Less sediment deposited in the lock approach channel, and deposition at the miter gate was about the same.

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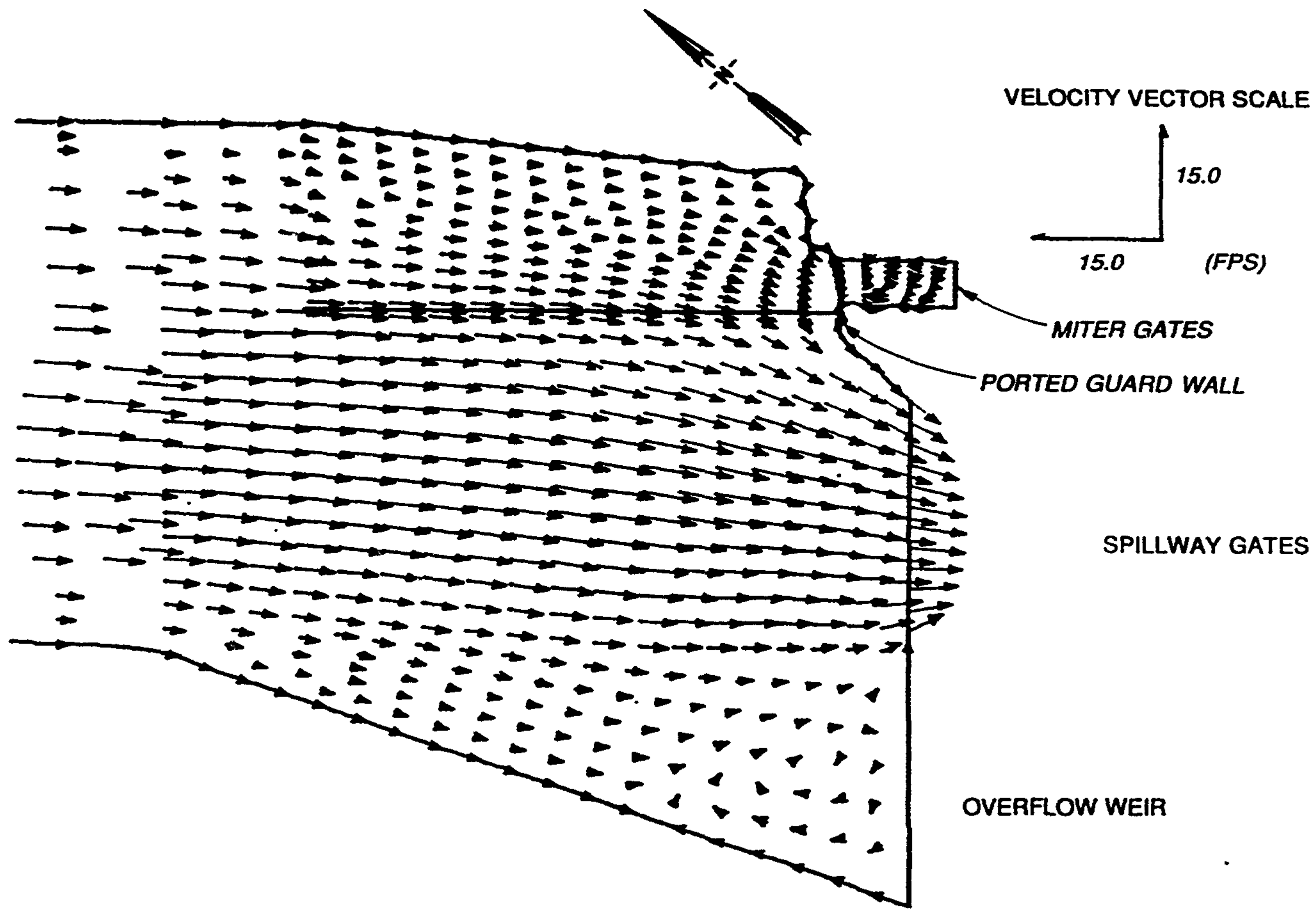
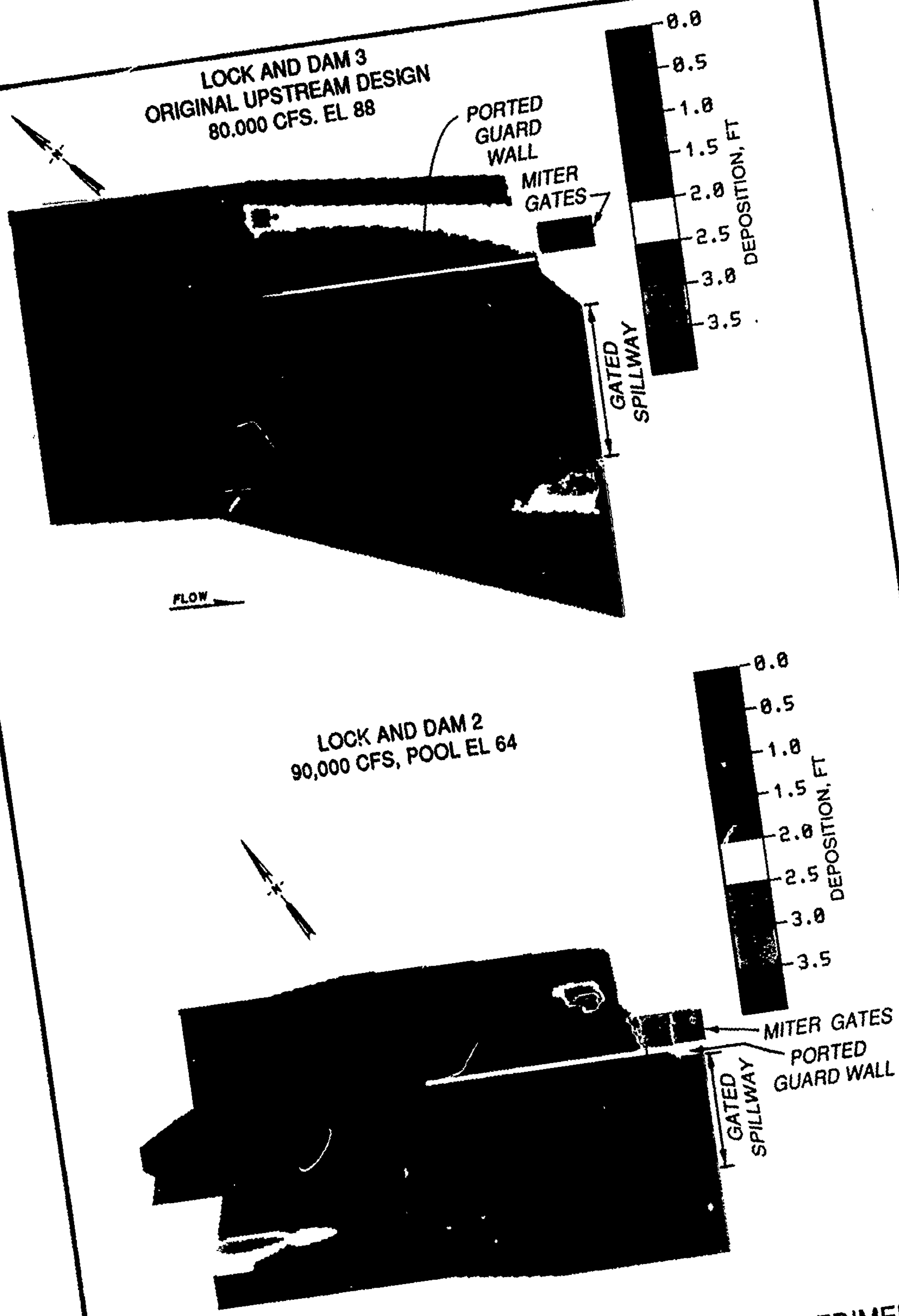
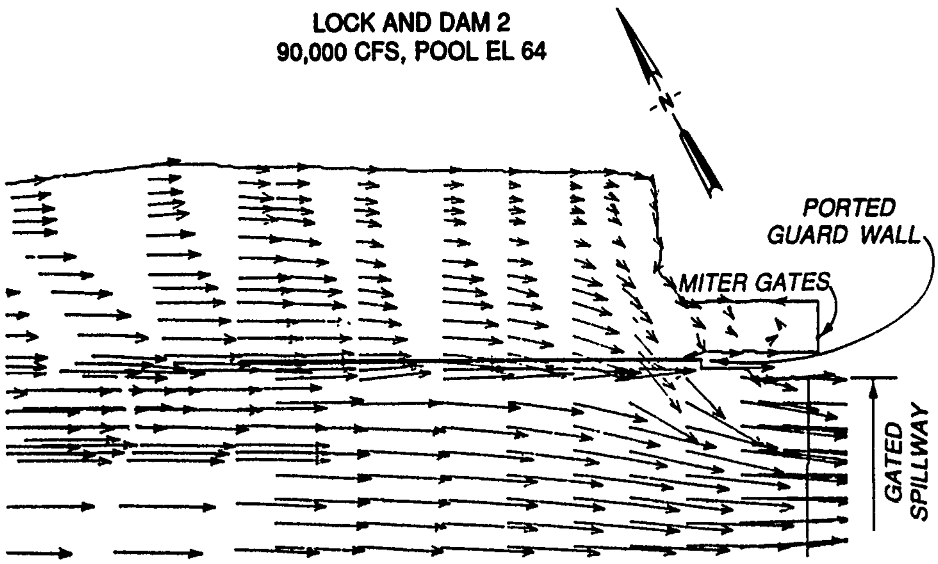
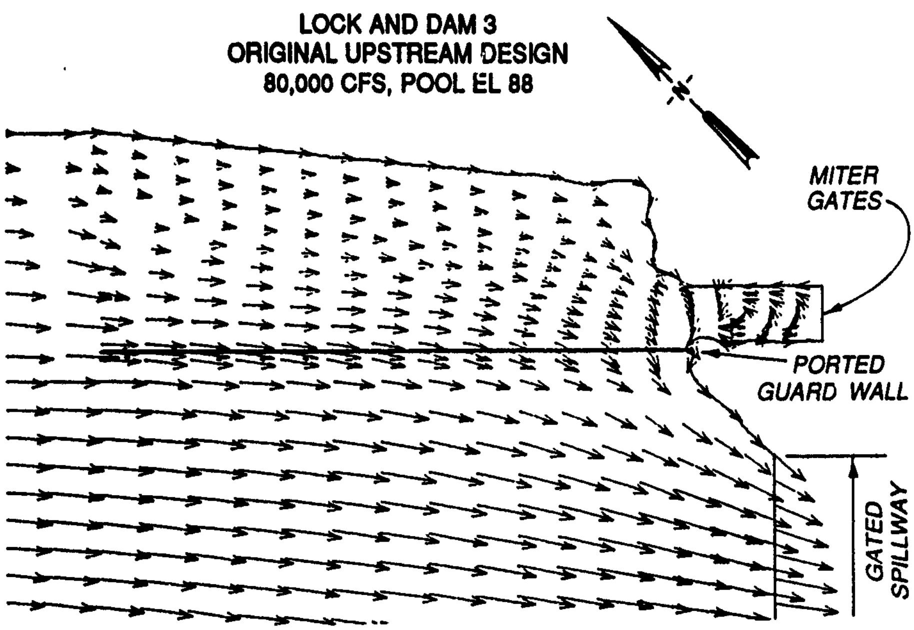


VELOCITY VECTORS
ORIGINAL UPSTREAM DESIGN
80,000 CFS
POOL EL = 88.0



DEPOSITION OF FINE SEDIMENT
10-DAY SIMULATION

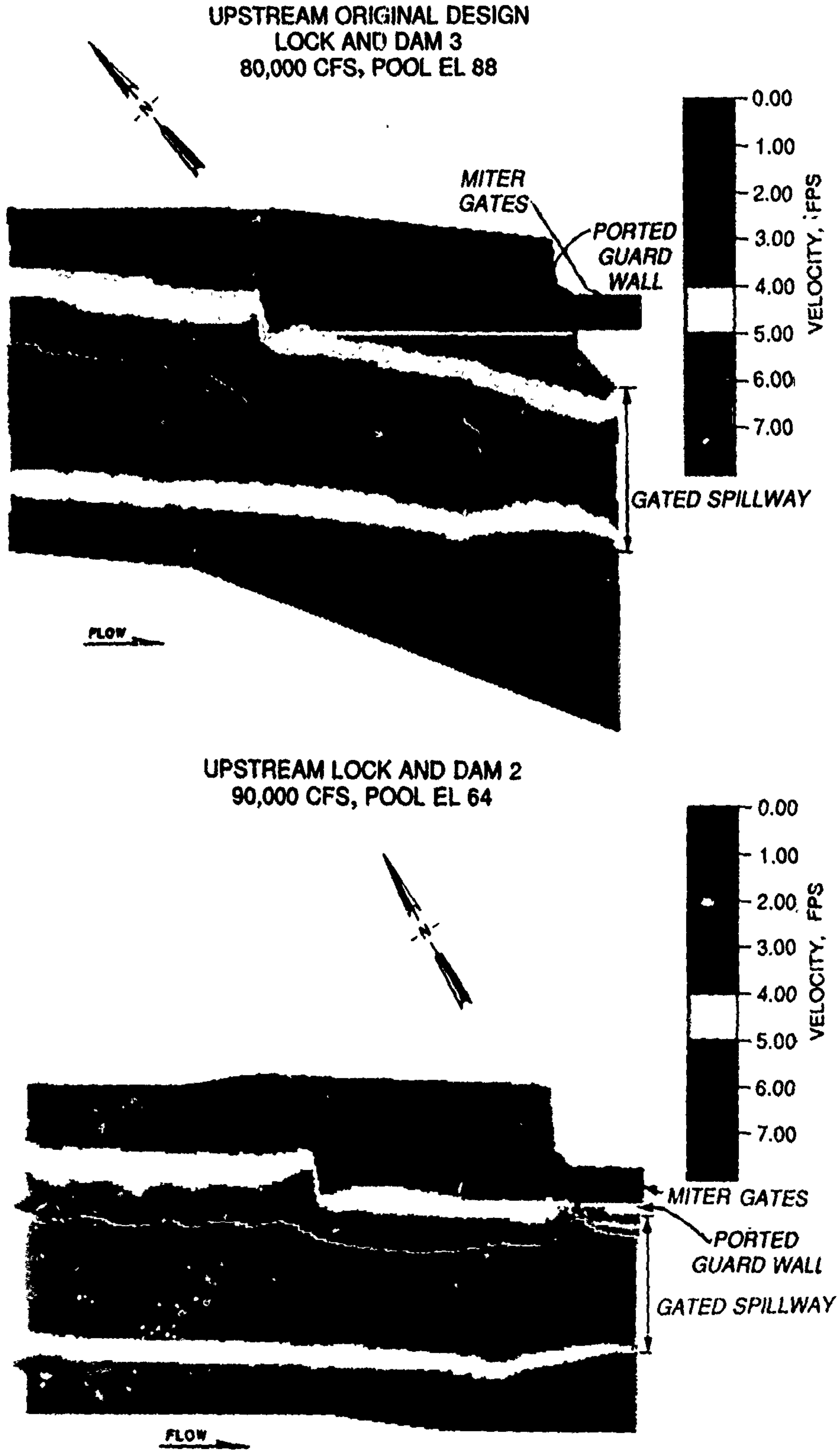




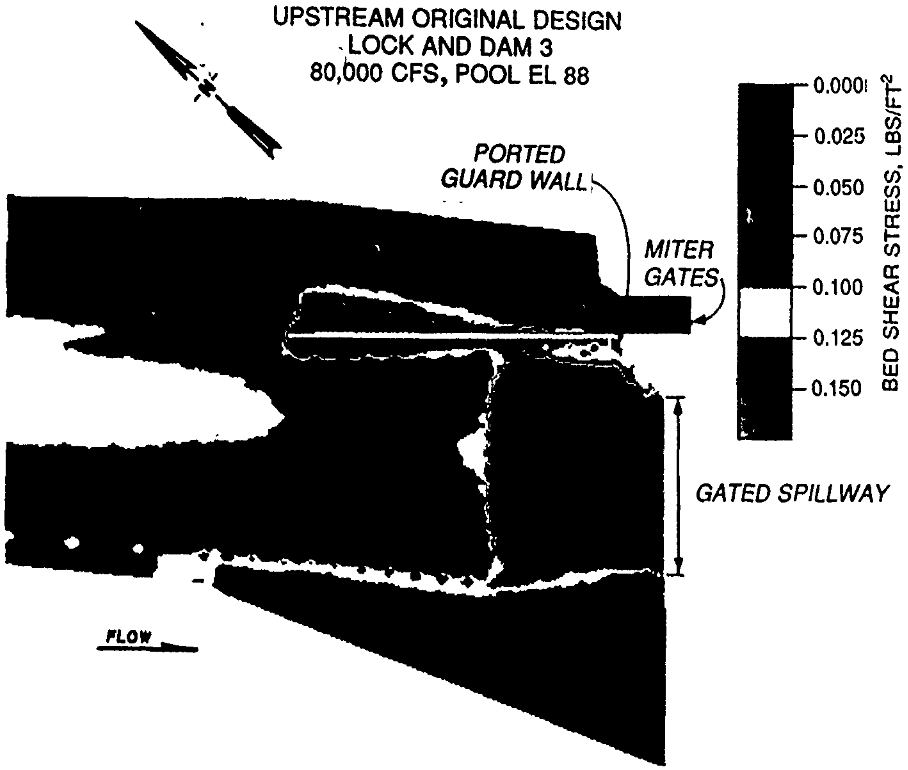
NOTE. SCALES ARE NOT THE SAME - DO NOT COMPARE VELOCITY MAGNITUDES

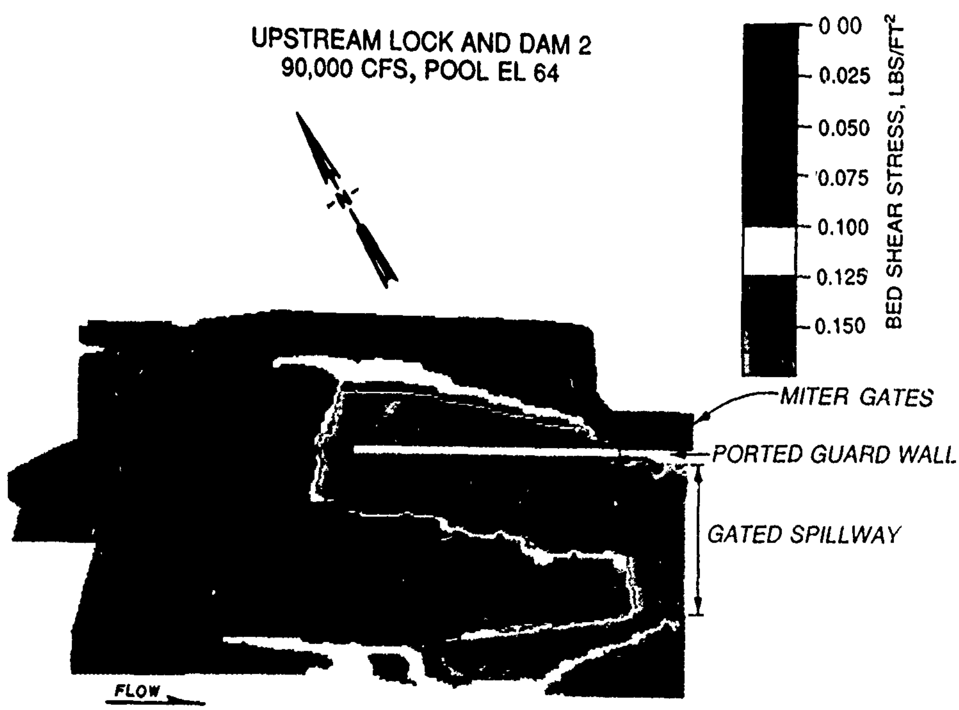
VELOCITY VECTORS

VICINITY OF GUARD WALL
LOCKS AND DAMS 2 AND 3

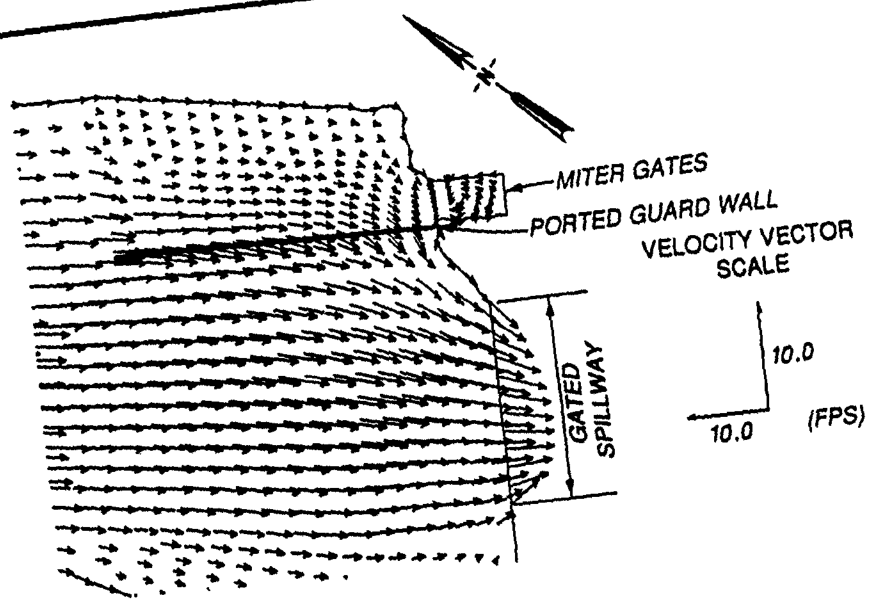


VELOCITY CONTOURS

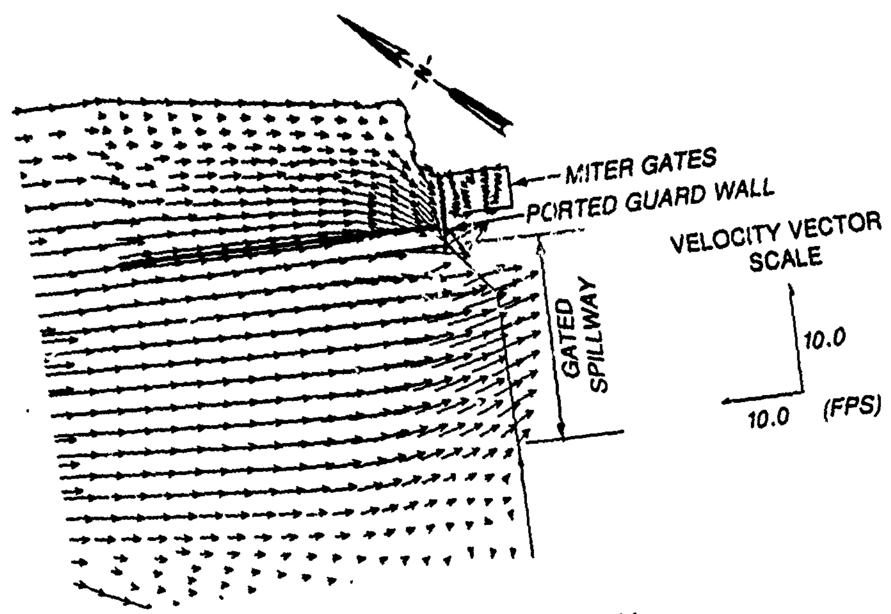




BED SHEAR STRESSES

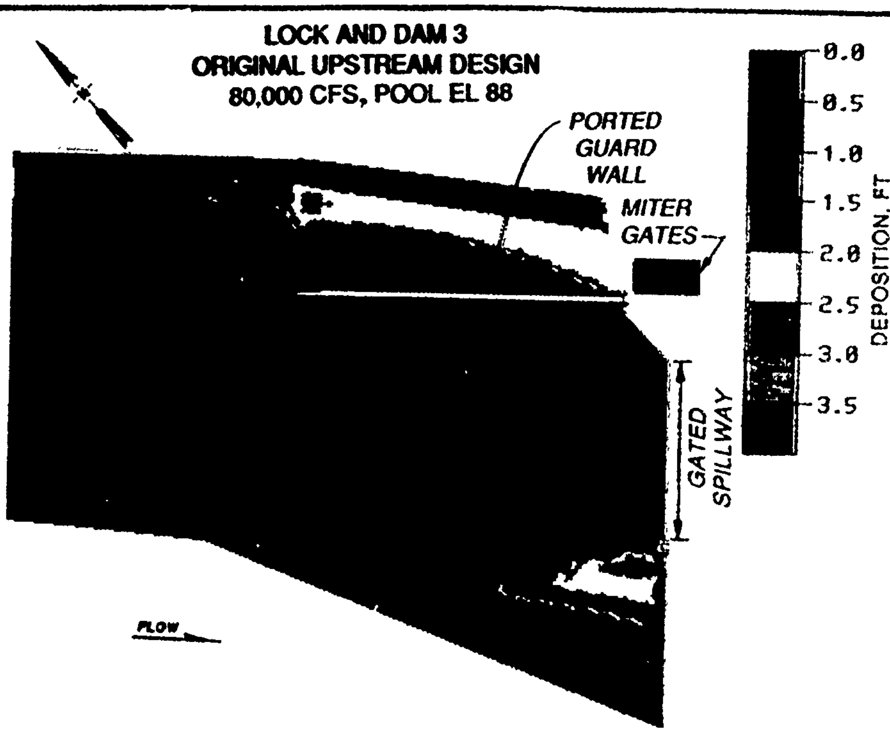


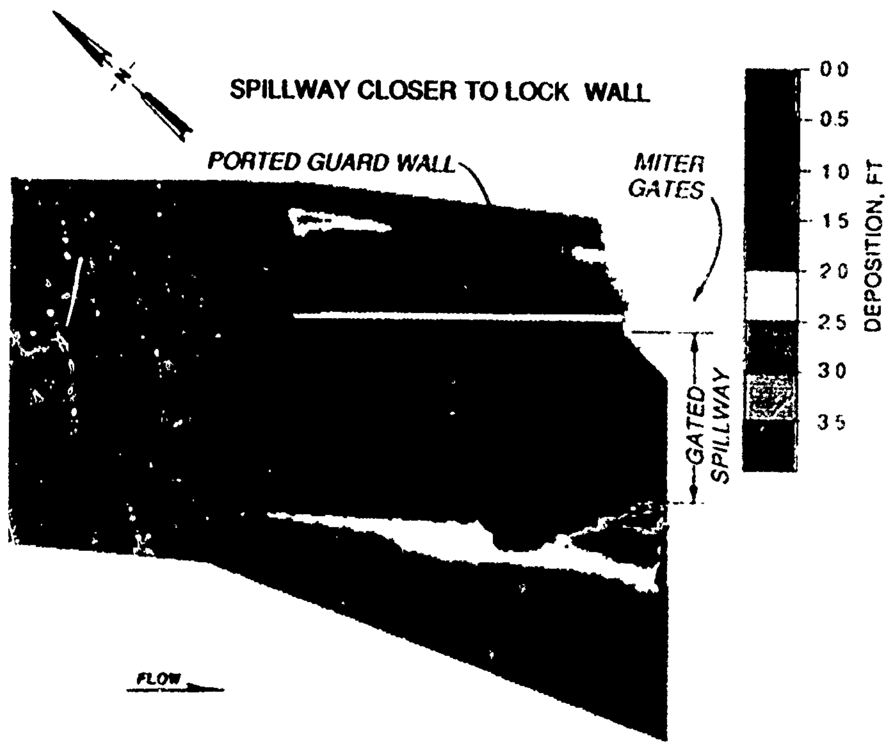
ORIGINAL DESIGN



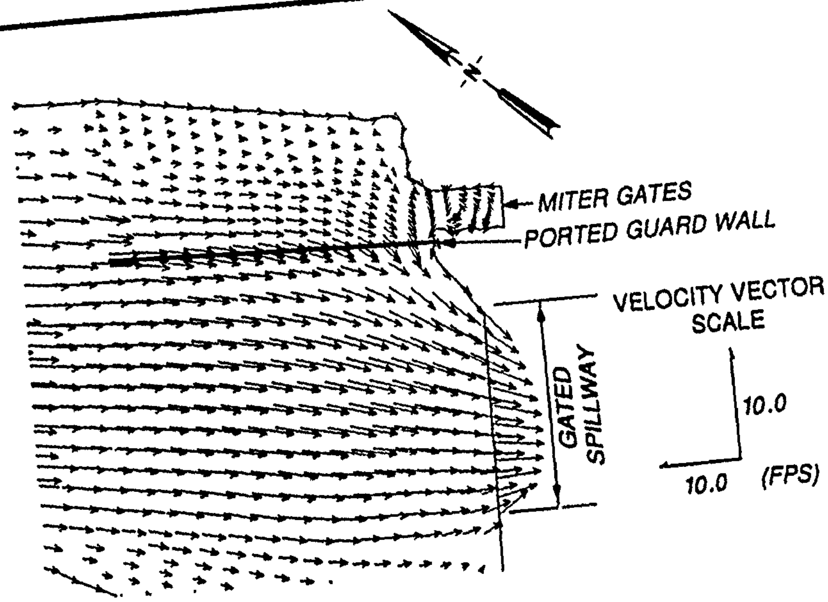
SPILLWAY CLOSER TO LOCK WALL

VELOCITY VECTORS
UPSTREAM LOCK AND DAM 3
EFFECT OF DISTANCE BETWEEN
LOCK WALL AND SPILLWAY
80,000 CFS, POOL EL 88





DEPOSITION OF FINE SEDIMENT
UPSTREAM OF LOCK AND DAM 3
EFFECT OF MOVING SPILLWAY
CLOSER TO LOCK WALL
10 DAY SIMULATION
80,000 CFS, 7.0 MG



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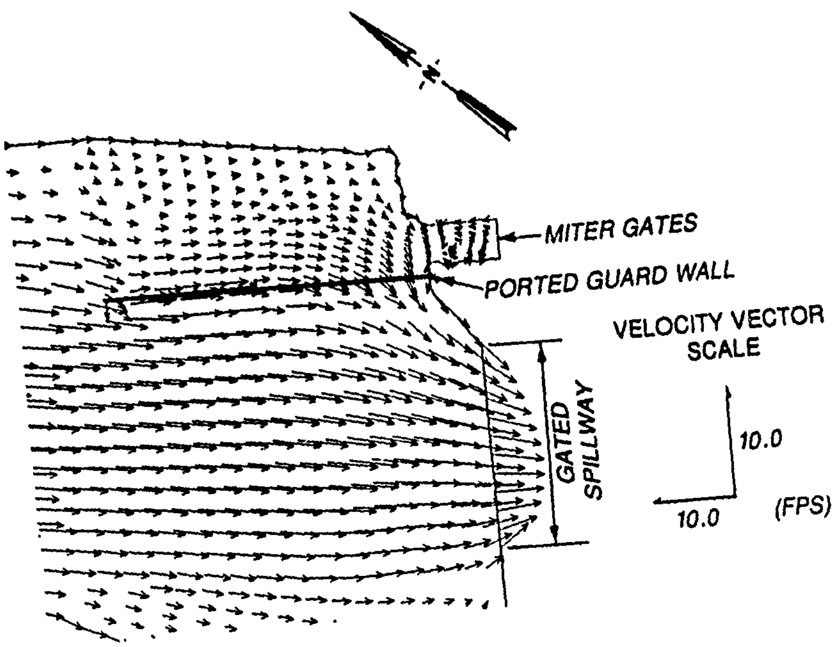
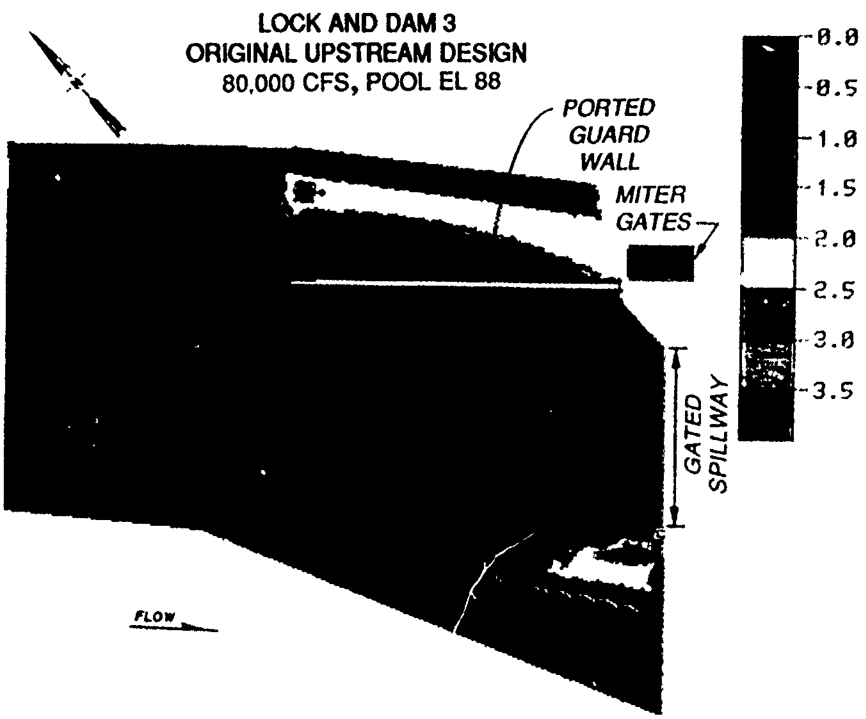
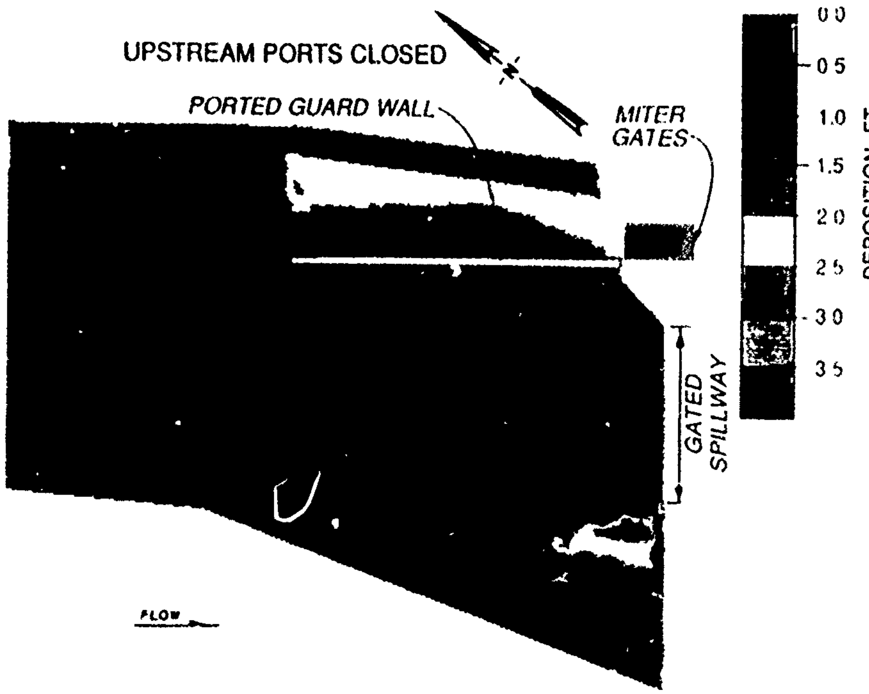


UPSTREAM PORTS CLOSED

VELOCITY VECTORS
UPSTREAM LOCK AND DAM 3
EFFECT OF CLOSING
UPSTREAM PORTS
80,000 CFS, POOL EL 88

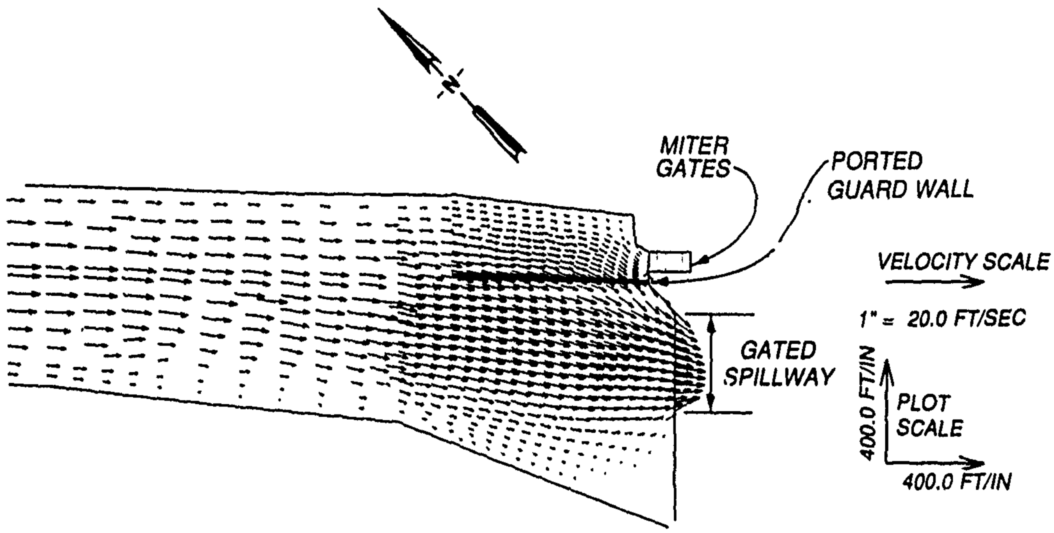




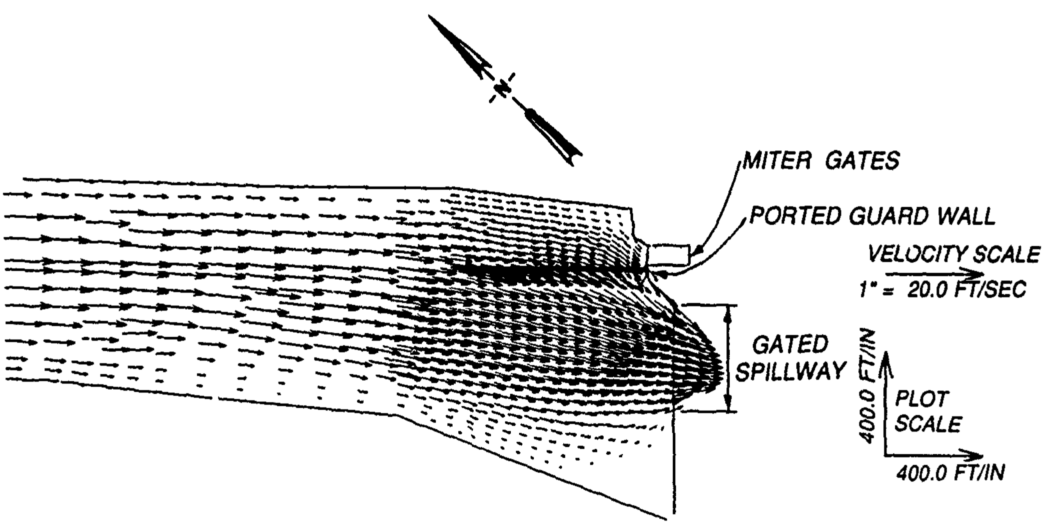
DEPOSITION OF FINE SEDIMENT
UPSTREAM OF LOCK AND DAM 3
EFFECT OF CLOSING UPSTREAM PORT

10 DAY SIMULATION
80,000 CFS, 72[illegible]

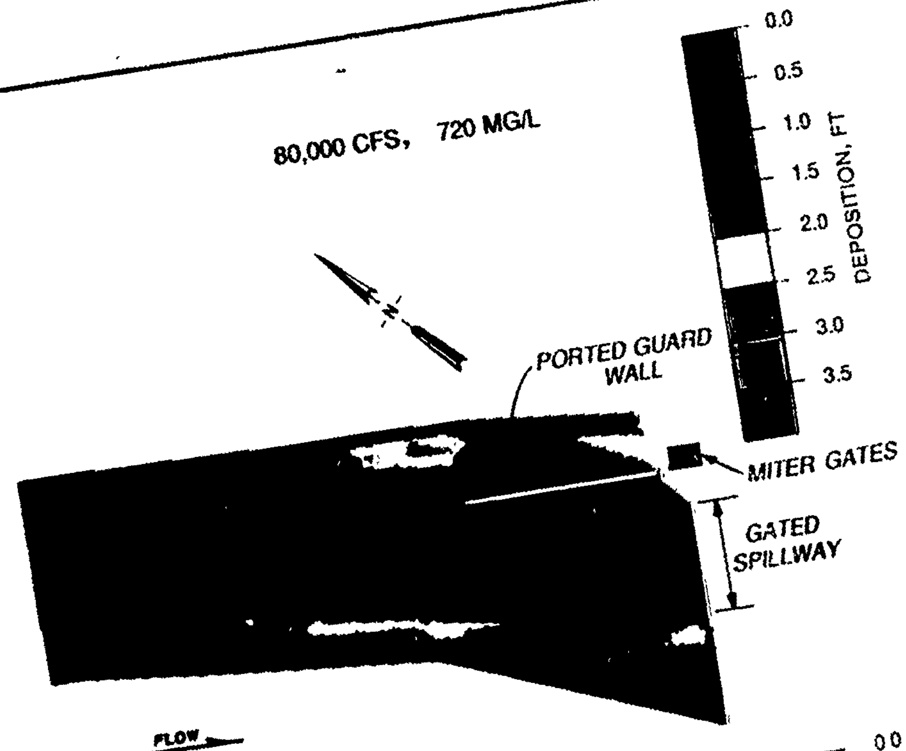
80,000 CFS, POOL EL 88

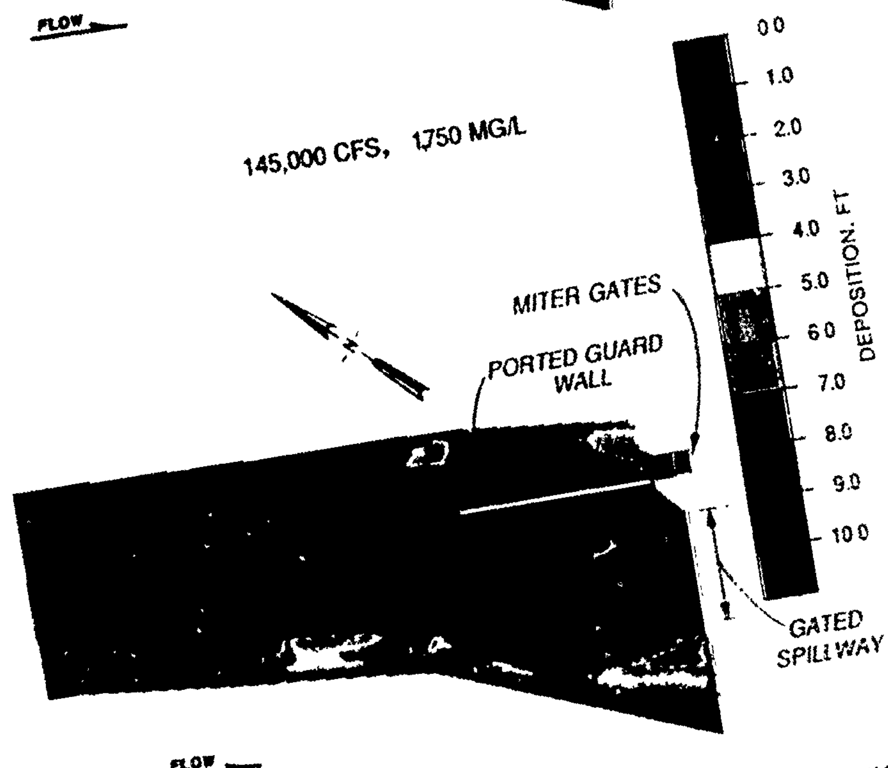


145,000 CFS, POOL EL 92.2



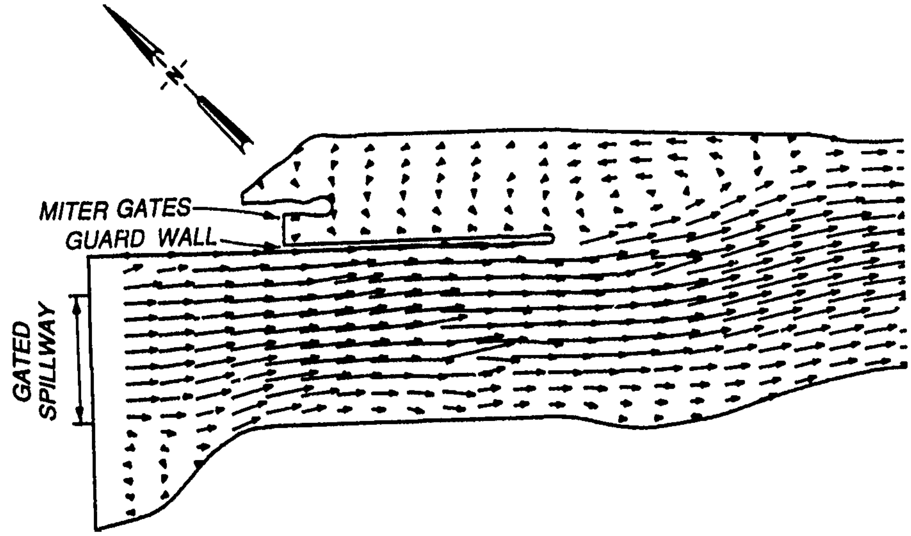
VELOCITY VECTORS
UPSTREAM OF LOCK AND DAM 3
PLAN G



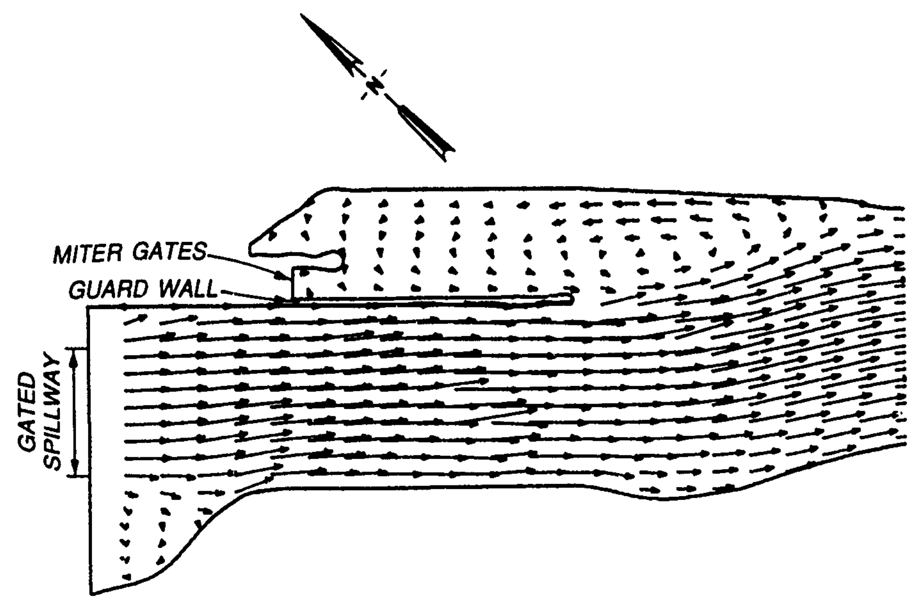


FINE SEDIMENT DEPOSITION
UPSTREAM OF LOCK AND DAM 3
PLAN G
10-DAY SIMULATION

PLATE

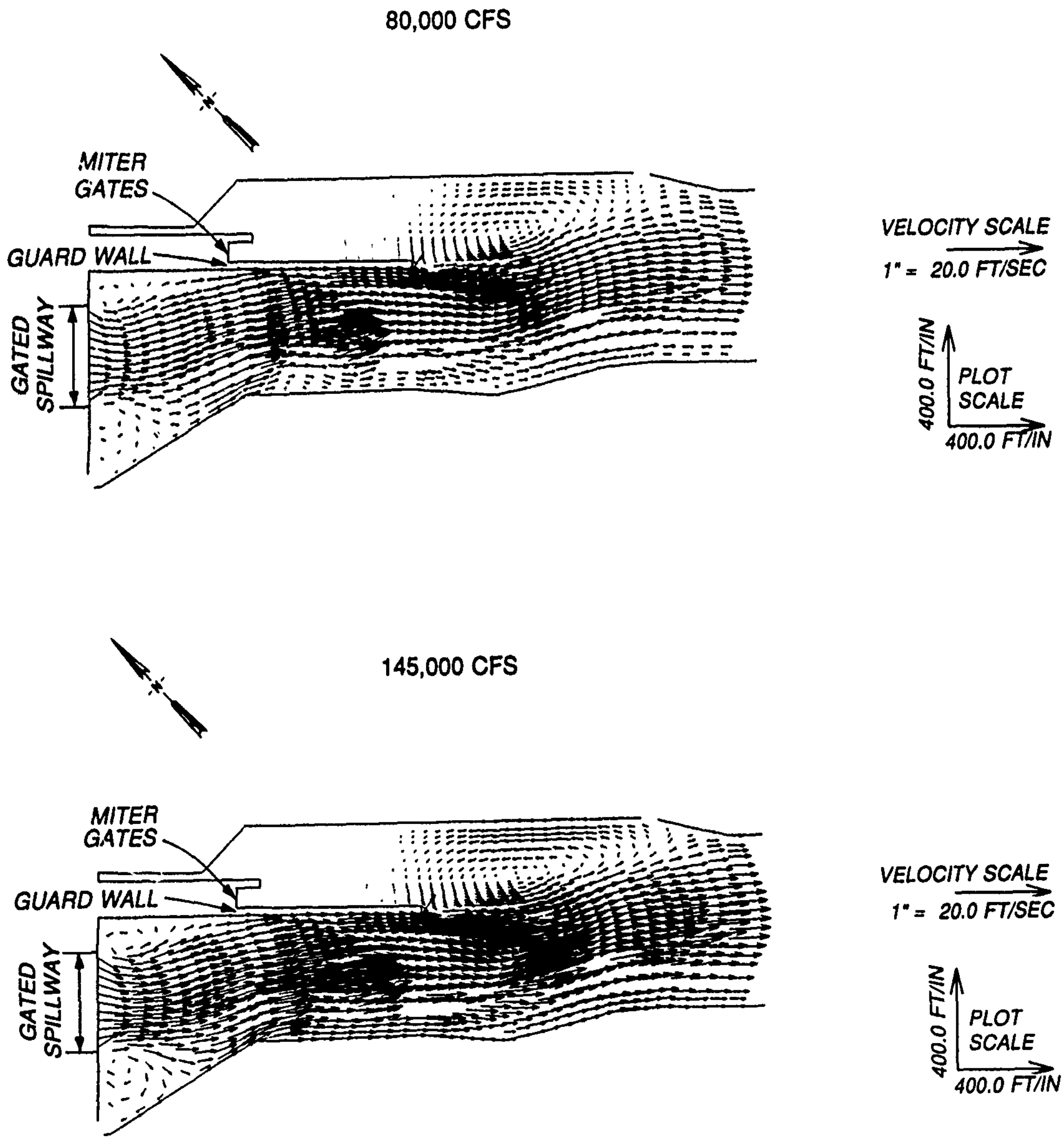


90,000 CFS

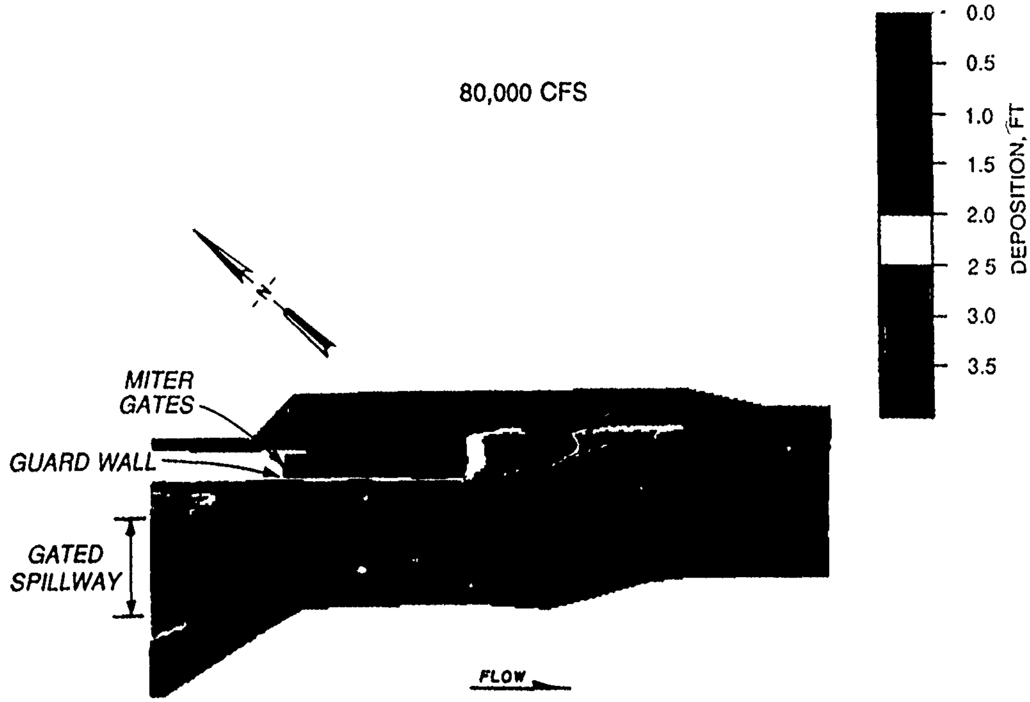


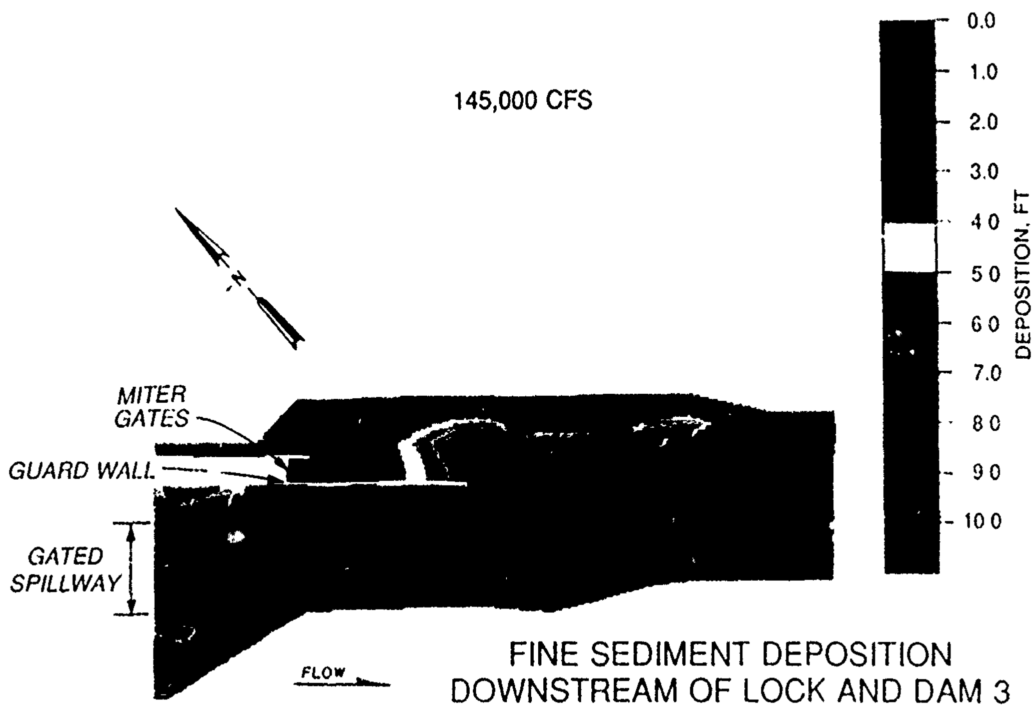
145,000 CFS

VELOCITY VECTORS
DOWNSTREAM LOCK AND DAM 3
PLAN B



VELOCITY VECTORS
DOWNSTREAM OF LOCK AND DAM 3
PLAN G





FINE SEDIMENT DEPOSITION
DOWNSTREAM OF LOCK AND DAM 3
PLAN G
10-DAY SIMULATION